MORPHOLOGY, AGEING AND ENGINEERING BEHAVIOUR OF SANDS

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CERTIFICATE

It is certified that the thesis work entitled *Morphology, Ageing and Engineering Behaviour of Sands* by JIMMY THOMAS has been carried out under my supervision, and that it has not been submitted elsewhere for a degree.

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Synopsis

The present work is an attempt to study the effect of morphology and ageing on some aspects of the engineering behaviour of sands. The term morphology refers to the form or shape of the grains. Morphology has a significant influence on the deformation and crushing of the grains, the geometrical arrangement of the particles and on the relative movements between grains and hence has a profound impact on the mechanical behaviour of sands. Ageing denotes the time dependent changes occurring in the structure of sands. In the case of sands, two categories of such phenomena are of interest. The first comprises the time dependent changes occurring in the structure of sands during their post-depositional history, under conditions of constant effective overburden stress. The second consists of certain time dependent processes observed to occur after disturbance of ground caused by pile driving or compaction. The effects of morphology and ageing are of great relevance in the interpretation of the results of in-situ tests.

In the present work some of these aspects have been investigated. Experiments were conducted on materials with a wide range of particle shapes to study the influence of morphology. Experiments have also been carried out to examine the outcome of secondary compression and cyclic pre-straining on the behaviour of sands in one-dimensional compression and direct shear. In addition, some of the available information on these topics has been critically evaluated and interpreted. The work presented in this thesis is organized in five chapters. In Chapter 1 the nature of the phenomena is described and their relevance to the solution of practical geotechnical engineering problems is highlighted. A detailed review of the information available on several aspects of morphology and ageing is presented in Chapter 2. The properties of the materials used in experimental work and the details of test procedures are described in Chapter 3. The significant results obtained are presented and interpreted in Chapter 4. The important conclusions are summarized in Chapter 5. Some suggestions for further research are also outlined.

The significant findings resulting from this work are the following. The

order of magnitude of the grain angularity of a sand can be estimated from information on e_{max}, C_u and D_{50}. Differences in behaviour caused by variations in the method of specimen preparation may be more pronounced in the case of angular sands in comparison to rounded sands. A fall cone test for the determination of e_{max} has been proposed. This procedure gives values which are practically same as those determined by the conventional method of pouring. A one-point variant of this method also leads to identical results. For most poorly graded, non-micaceous sands, there is a strong correlation between e_{max} and e_{min}. This linear relationship may be used to obtain a reasonably accurate estimate of e_{min}.

Differences in particle angularity can result in changes in ϕ_{cv} of the order of 6°. The higher values of ϕ_{cv} in the case of angular sands are likely to be due to a larger contribution from particle rearrangement, and this appears to be closely related to the more non-uniform distribution of local void ratio. For a given magnitude of the confining stress, $e \tan \phi'$ for a sand is a constant. The value of this constant decreases as roundness increases. Based on this concept, a relationship for estimating the angle of shearing resistance has been proposed. This relationship may be used to demonstrate the influence of morphology on ϕ'. An analysis of the fall cone test for sands has been presented and this has been used to compute values of N_γ from the results fall cone tests. In the case of a wellrounded sand the relationship between experimentally determined values of N_γ and estimated angles of shearing resistance is in close agreement with a theoretical relationship proposed by Meyerhof (1961). But for an angular sand the agreement is poor. The range of initial states corresponding to different types of undrained shear response of saturated sands, is to a large extent controlled by the particle shape. Morphology also has a significant influence on the relationships between some of the parameters characterizing the undrained shear behaviour.

Roundness has a significant influence on modulus number m_0. It has also some effect on m_2, but m_1 appears to be independent of roundness. Morphology affects the nature of the strain-time relationships of sands in one-dimensional compression. However, the effect of particle shape on the value

C_α/C_c appears to small. Grain angularity leads to significant differences in the magnitude of strains induced and the nature of the hysteresis loops developed during one-dimensional cyclic loading of sands.

The effects of equal amounts of strain induced by cyclic pre-straining within a sufficiently small stress range and secondary compression, on the one-dimensional stress-strain and strain-time response of sands are practically identical. But the outcome of overconsolidation is significantly different. Secondary compression and cyclic pre-straining causes the behaviour of sands during shear to be less contractive. Cyclic pre-straining using a sufficiently narrow stress range, may be employed to simulate the effects of secondary compression in the laboratory, because of the observed similarity between their consequences. The large time-dependent increase in penetration resistance reported to occur in sand-fills after placement, cannot be attributed to the effects of secondary compression alone. The available evidence does not appear to justify the hypothesis that, secondary compression is the predominant mechanism which causes the large time-dependent increase in post densification penetration resistance.

Nomenclature

C_c	Compression index
C_D	Parameter which reflects densification due to disturbance
C_r	Recompression index
C_u	Coefficient of uniformity
C_α	Secondary compression index
D	Particle size
D_{10}	Particle size corresponding to 10% finer
D_{50}	Particle size corresponding to 50% finer
D_{60}	Particle size corresponding to 60% finer
d	Penetration measured in fall cone test
d_0	Penetration at which the ultimate bearing capacity of the cone is equal to the total weight of cone assembly
d_I	Intermediate particle dimension
d_L	Longest particle dimension
d_S	Shortest particle dimension
E	Deformation modulus
E_{max}	Maximum deformation modulus
e	Void ratio
e_0	Void ratio at steady state for $P' = 0$
e_{cr}	Void ratio at critical state
e_i	Void ratio corresponding to $\sigma'_v = 0$
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
e_r	Reference void ratio for computing state index
e_s	Void ratio at quasi-steady state
e_{ss}	Void ratio at steady state
G	Shear modulus
G_{max}	Maximum shear modulus
$G_{max(t)}$	Maximum shear modulus at time t during ageing
$G_{max(t_R)}$	Maximum shear modulus at reference time t_R
G_s	Specific gravity

g	Acceleration due to gravity
I_c	Grain contact index
I_D	Relative density
I_R	Relative dilatancy index
I_s	State index
K	Cone factor
K_0	Coefficient of earth pressure at rest
$K_{0,nc}$	Value of K_0 for normally consolidated soils
$K_{0,oc}$	Value of K_0 for overconsolidated soils
K_{0p}	Value of K_0 at end of primary consolidation
$K_{0,t}$	Value of K_0 at time t during secondary compression
K_{0,t_R}	Value of K_0 at reference time t_R
M	Constrained modulus
M_p	Constrained modulus at end of primary consolidation
m	Modulus number
m_0	Modulus number corresponding to $I_D = 0$
m_1	Exponent indicating the rate of change of M with vertical stress
m_2	Exponent indicating the rate of change of M with relative density
N	Standard penetration test blow count
$(N_1)_{60}$	Corrected N value corresponding to 60% free fall energy and vertical effective stress of 100 kPa
N_G	Normalized rate of increase of G per log cycle of time
N_γ	Bearing capacity factor
n_{max}	Maximum porosity
n_{min}	Minimum porosity
OCR	Overconsolidation ratio
P	Mean principal normal stress $= (\sigma_1 + 2\sigma_3)/3$
P'	Mean principal normal effective stress $= (\sigma_1' + 2\sigma_3')/3$
P_c'	Value of P' at end of consolidation
P_f'	Value of P' at failure
P_{ms}'	Value of P' corresponding to the intersection of $e = e_0$ line with the isotropic compression line

P'_p	Value of P' corresponding to the peak deviator stress
P'_s	Value of P' at quasi-steady state
p	Exponent in Hardin's model for one-dimensional compression
Q	Empirical constant in relative dilatancy index
q	Deviator stress $= \sigma_1 - \sigma_3$
q_c	Cone tip resistance
$q_{c,nc}$	Value of q_c for normally consolidated soil
$q_{c,oc}$	Value of q_c for overconsolidated soil
$(q_c)_R$	Value of q_c at reference time t_R
q_{ss}	Value of q at steady state
R	Value of roundness for a sand
R^*	Empirical constant in relative dilatancy index
r_c	Initial stress ratio
S_{1Dmax}	Hardin's one-dimensional stiffness coefficient
SF	Shape factor
s_p	Normalized peak undrained shear strength
s_{us}	Normalized residual undrained shear strength
t	Time
t_p	Time corresponding to end of primary consolidation
t_R	Reference time
v	Velocity of motion of fall cone at any instant of time
W	Total weight of the fall cone assembly
z	Depth of penetration of fall cone at any instant of time
α	Empirical constant relating e_{max} and e_{min}
β	Apex angle of fall cone
γ	Unit weight of soil
Δe	Increment of void ratio
ΔG_{max}	Increment of G_{max} per log cycle of time
$\Delta\sigma'_v$	Increment in vertical effective stress
ϵ_a	Axial strain
σ	Normal stress
σ'_0	Initial isotropic effective confining stress

σ'_1	Major principal effective stress
σ'_3	Minor principal effective stress
σ_a	Atmospheric pressure
σ'_h	Horizontal effective stress
σ'_v	Vertical effective stress
σ'_{v0}	Vertical effective stress during secondary compression
σ'_{vmax}	Past maximum vertical effective stress experienced by soil
τ	Shear stress
ϕ	Angle of shearing resistance
ϕ_{cv}	Constant volume friction angle
ϕ_μ	Angle of inter-particle sliding friction
ϕ'	Effective angle of shearing resistance
ϕ'_f	Value of ϕ' corrected for the effect of dilatancy
ϕ'_{ss}	Steady state angle of shearing resistance
χ	Empirical constant which reflects the effect of overconsolidation on q_c
ψ	State parameter

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Chapter 1

Introduction

In the past, the engineering behaviour of sands has not been as exhaustively investigated as that of clays, probably because sands were rarely considered as troublesome materials. Design methods were usually based on simple in-situ tests and past experience. But recently attempts are being made to devise more sophisticated techniques for predicting the response of sands to applied loads. These have been primarily motivated by the need for more accurate predictions of liquefaction potential and foundation displacements. The development of rational procedures for predicting the response of sands, requires a thorough understanding of the various factors which govern the engineering behaviour of sands.

Sands are particulate materials and hence various grain characteristics like mineralogy, morphology and particle size distribution have a profound influence on mechanical behaviour. The term morphology refers to the form or shape of the grains. Even though the behaviour of sands is usually studied at the phenomenological level, it is clear that the macroscopic response of sands is the result of interactions at the particulate level. The effects of particle morphology are manifested at different levels. Firstly, the deformation, modification and crushing of the grains are strongly influenced by their shape. Secondly, particle shape has a marked effect on the geometrical arrangement of the grains (fabric) which is an important variable governing the response of sands. Lastly, morphology has a pronounced impact on the relat-

ive movements between particles and hence on compressibility and dilatancy. Therefore for a correct understanding of the behaviour of sands, it is very important to consider the effects of morphology. A major objective of the present work is to highlight the importance of morphology and to examine its effect on the fabric and strength and deformation characteristics of sands.

A particular sand with a given set of particle characteristics can exist in different states. The response of a sand to imposed loads would be controlled by the state variables characterizing the geometrical arrangement of the grains and the forces acting between them. The important state variables are—the orientation of the particles and contact normals, the mean and standard deviation of void ratio, the type of grain contacts, the nature and degree of cementation, the state of stress and the stress and strain history. In the past, the behaviour of sands was assumed to be mainly a function of void ratio (or relative density), confining stress and, if applicable, mechanical overconsolidation. However, experience gained over the last few decades indicates that there are other factors which exercise a significant influence on the response of sands. The phenomenon of ageing which leads to significant changes in the structure and strength and deformation characteristics of sands, is one such important factor and is the second theme of the present thesis.

In geotechnical engineering the term ageing is used to denote three classes of phenomena. These are:

- The changes occurring in the micro-structure and water content distribution of samples during the period between retrieval and testing.
- The time dependent changes in the structure of soils subsequent to deposition (ageing under geo-static conditions).
- Certain time dependent processes frequently observed to occur after disturbance of ground (ageing after disturbance of ground).

Since the first aspect is normally important in the case of fine-grained soils only, it is not investigated in this work. In the case of sands ageing under geo-

static conditions and also after ground disturbance are of interest and hence an attempt was made to examine some aspects related to these phenomena.

The time dependent changes occurring in the state of a sand during its post-depositional history, under conditions of constant effective overburden stress, are collectively called as ageing. During this period there is no significant change in relative density and vertical effective stress which are the two state variables usually considered to be the most important. But there are indications that ageing induces changes in some of the other state variables, which often lead to significant improvements in the strength and deformation characteristics of sands. It has also been observed that after the disturbance of ground caused by pile driving or, densification by blasting, heavy tamping or vibro-compaction, time dependent changes may occur as indicated by increase in pile capacity or penetration resistance with time. Such phenomena are also usually referred to as ageing. In both the cases the major issues of interest are the mechanisms of ageing, their effects on mechanical behaviour and the development of techniques to predict these changes. In the present work some of these aspects have been studied.

In order to highlight the relevance of morphology and ageing to the solution of practical geotechnical engineering problems involving sands, a brief description of the philosophy of design is of interest. Even though soils are particulate materials, usually they are idealized as a continuum and the response is predicted using the laws of continuum mechanics. The material properties are introduced through appropriate constitutive relationships. The parameters of the constitutive models are normally evaluated from the measured response of samples which are tested in the laboratory. Thus the continuum approach treats the response of soils at the phenomenological level only and hence is also called the phenomenological approach. In spite of the fact that the particulate nature of soils is not taken into account explicitly, the phenomenological approach can yield excellent results, provided, the response of the tested sample is identical to that of an element of soil in-situ. This condition is satisfied only if the samples are truly undisturbed and the test procedures are free from errors. Under such circumstances a purely phe-

nomenological approach may be sufficient, and perhaps it is not necessary to consider the behaviour at the particulate or structural level.

In the case of sands, obtaining undisturbed samples is often very difficult and prohibitively expensive and this necessitates extensive reliance on in-situ tests for the determination of material parameters. A major problem with in-situ tests is that they do not measure any basic constitutive parameter. In general there are three approaches towards the use of the results of penetration tests in design (Jamiolkowski et al. 1988). In the first category of methods, the results of in-situ tests are directly related to the response under imposed loads. Examples of this approach are—Terzaghi and Peck's charts for allowable bearing pressures, Burland and Burbidge's (1984) method for predicting settlements of foundations and the evaluation of liquefaction potential from penetration resistance. The second approach entails the formulation of a mathematical model for the response of the sand during the test. With appropriate assumptions regarding the boundary and drainage conditions and constitutive relationships, the inverse problem of computing the material parameters from the measured response is solved. This approach is reliable in the case of pressuremeter and seismic tests, but in the case of penetration tests the results are not very satisfactory. In the third approach, empirical correlations are established between the results of in-situ tests carried out in large calibration chambers and the required soil characteristics determined from laboratory tests. Typical examples are the widely used correlations between deformation moduli and cone tip resistance.

Thus in the case of penetration tests which are the most extensively used in-situ tests, the measured penetration resistance is either directly related to the response under imposed loads (settlement of foundations, liquefaction potential) or correlated to selected engineering properties like the angle of shearing resistance and deformation moduli. It may be noted that while these purely empirical correlations may be valid for the conditions under which they were established, there is no fundamental reason as to why such relationships between apparently different aspects of behaviour should be universally applicable to all sands. Therefore it is important to identify the

factors which govern the validity and the domain of applicability of these relationships.

The different aspects of the behaviour of sands like penetration resistance and deformation moduli, are functions of the grain characteristics and the state variables. Hence any empirical correlation between penetration resistance and deformation moduli, is only a reflection of the fact that these variables are comparable functions of the same set of factors. If the effect of various grain characteristics and state variables on penetration resistance is comparable in magnitude to that on deformation moduli, then an empirical correlation between penetration resistance and deformation moduli may be applicable to all sands. But if the effect of any factor on one variable is significant and on the other is negligible, it is clear that an empirical correlation between these variables which was developed for a particular sand, is not be applicable to all sands. There are indications that the outcome of morphology and ageing on penetration resistance is not the same as that on other aspects of behaviour. Hence consideration of these phenomena is of great relevance to the interpretation of the results of penetration tests.

The results of cone penetration tests carried out in calibration chambers show that in general, sands of low compressibility exhibit higher cone tip resistance in comparison to sands of high compressibility. Hence it is likely that under comparable conditions, penetration resistance of angular sands is lower than that of rounded sands. As the strength and deformation characteristics are assumed to be proportional to the penetration resistance, this would suggest that at the same relative density and confining stress angular sands would have lower stiffness and liquefaction resistance. However, such an implication is not substantiated by the available evidence. A possible reason for the lower penetration resistance of angular sands could be the grain modification and crushing induced by the very high stresses acting in the vicinity of the cone tip. The results of model penetration tests reveal that at very low levels of confining stress angular sands have higher penetration resistance. For most problems in geotechnical engineering where the operating stress levels are not sufficient to cause particle modification and

crushing, there is no reason to expect that the deformation moduli of angular sands would be significantly lower than that of rounded sands. There is also some evidence that angular sands have higher resistance to liquefaction than rounded sands. Hence if correlations which are established on the basis of tests on rounded sands are used to predict the behaviour of angular sands, the results may not be satisfactory.

The effects of ageing are also of great relevance to the interpretation of the results of penetration tests. Calibration chamber and laboratory tests are normally performed on freshly deposited specimens which are tested immediately after consolidation under the applied confining stresses. On the other hand, natural deposits of sand would have existed under the current overburden stress for several thousands of years and hence would have undergone ageing. There are reasons to believe that the effects of ageing on penetration resistance may not be as pronounced as that on deformation moduli and undrained shearing behaviour. Therefore correlations developed on the basis of tests on freshly deposited specimens may lead to considerable underestimation of the stiffness and liquefaction resistance of aged sands. Ageing after densification of ground is important for the evaluation of the efficacy of the densification programme and also for the selection of design parameters.

The preceding description of the methodology of solution of geotechnical engineering problems involving sands shows that, for a realistic assessment of the in-situ behaviour of sands, it is extremely important to consider the effects of morphology and ageing. Design criteria are expected to become more stringent in the future and hence there is a need to evolve rational and reliable methods for predicting the behaviour of sands. Over the years, spectacular developments have taken place in geotechnical analysis and testing procedures. But the capabilities of determining strength and deformation characteristics which accurately reflect the in-situ response of sands, have not developed to the same extent. Progress in this area would depend to a large extent on the advances made in the interpretation of the results of in-situ tests. For this it is necessary to consider the effects of factors which

are significant but have not been accorded due importance in the past. The present work is a step in this direction where an attempt has been made to examine some aspects of the relationship of morphology and ageing to the engineering behaviour of sands.

Efforts have been made to study the influence of morphology, especially particle angularity, on the fabric and strength and deformation properties of sands. Considerable amount of data on the behaviour of sands with different morphological characteristics is available in literature. An important objective of the present work is to interpret some of these data to facilitate improved understanding. Experiments have been conducted on materials with a wide range of particle shapes to study the effect of morphology on several facets of behaviour. Some aspects of ageing have been examined by conducting experiments and also by interpretation of published data. The effects of secondary compression and cyclic pre-straining have been investigated by carrying out oedometer and direct shear tests. Some issues related to the mechanisms of ageing have been discussed on the basis of available information.

A detailed review of the literature on the earlier research on morphology and ageing is presented in Chapter 2. The characteristics of the materials used and the procedures of the experiments carried out in the present work are described in Chapter 3. The important results obtained in the course of this study are presented and interpreted in Chapter 4. The major conclusions drawn are summarized in Chapter 5. Some suggestions for further research in these areas are also outlined.

Chapter 2

Review of Literature

In this chapter a comprehensive review of the earlier research on various aspects of morphology and ageing and their influence on the engineering behaviour of sands is presented. For the sake of completeness, the information available on this subject is examined in detail.

2.1 Morphology and Engineering Behaviour of Sands

In order to quantify the effect of morphology the first requirement is its characterization. Morphology influences the behaviour of sands, not only directly through the shape of individual grains, but also indirectly through its control on fabric. In most problems of geotechnical engineering, the main issues of interest are the strength and deformation properties. Therefore the available information on the characterization of morphology and the relationship between morphology and the fabric and strength and deformation behaviour of sands, is reviewed in this section.

2.1.1 Characterization of Morphology

The term morphology refers to the shape or form of an object. It is difficult to define these terms precisely, but one intuitively understands what these really denote. Following Beddow (1980) morphology of a particle may be defined as the recognized pattern of relationships among all the points which constitute the external surface of the particle. Morphological characteristics of sands are a function of a large number of factors. According to Krumbein and Pettijohn (1938) morphology of sediments are controlled by the following factors—original shape of the fragment, the structure of the fragment as cleavage or bedding, the durability of the material, the nature of the geologic agent, the nature of the action to which the fragment is subject and the violence of this action, and the time or distance through which this action is extended. Considering the multiplicity of these factors and their nature, natural sands may be expected to have morphological characteristics that are complex and diverse and hence morphological characterization of these materials presents many problems.

Characterization of morphology comprises two principal tasks. These are formulation of quantitative indices or measures that are relevant to the area of interest and the development of reliable and economic procedures for measurement. Particulate materials are encountered in many disciplines—geology, civil engineering, powder metallurgy and in many manufacturing and material handling operations that deal with products in the form of food grains, tablets and capsules. While developments in morphological characterization have taken place in many of these fields, geotechnical engineers have borrowed most of their concepts and methods from sedimentary petrologists, probably because of their common interest in the same materials.

Sedimentary Petrologist's Approach

Over the years sedimentary petrologists have developed many concepts and methodologies for the characterization of the morphology of sedimentary particles. The evolution of these ideas is discussed by Krumbein and Pettijohn (1938), Pettijohn (1957), Griffiths (1967) and Blatt et al. (1972). The

following description essentially is adapted from these references.

Three Aspects of Morphology

In the work of sedimentologists, three aspects of morphology are highlighted. These are sphericity, roundness and surface texture.

Sphericity is a measure of the degree to which the shape of a particle approaches that of a sphere. This measure was proposed by Waddel (1932). He suggested that ideally sphericity may be defined as the ratio of the surface area of a sphere with the same volume as that of the particle and the actual surface area of the particle. But in view of the difficulties involved in the measurement of surface area, a simpler alternative was suggested, wherein sphericity is defined as the ratio between the diameter of a sphere with the same volume as that of the particle and the diameter of the circumscribed sphere. In general sphericity indicates the overall shape of the particle. A sphere has a value of 1 and all other shapes have values less than 1. Usually sphericity is computed from three linear dimensions measured on the projected image of the particle.

Roundness (R) is a measure of the sharpness of the corners and edges of the grains. It therefore refers to features of the grain surface which are on a larger scale than those classified as surface texture but still smaller than the overall dimensions of the grain. Wadell (1932) defines roundness as the ratio of the average radius of curvature of the corners to the radius of the largest inscribed circle. Because of the difficulties associated with the determination of R based on measurements on projected images of the grains, the common procedure is to estimate roundness from a visual comparison of the images with standard images of grains having known values of R. Powers (1953) prepared such a set of images and also proposed a scale for roundness in which the particles are classified into different classes ranging from the very angular to the wellrounded. Based on Powers' procedure Youd (1973) developed a method for arriving at an average value of roundness for a sand. He separated the sand into several fractions by sieving and from each sieve fraction selected at random at least about 50 particles. By comparing the

projected image of the grains in a microscope with the standard set of images he assigned each particle to one of Powers' roundness classes. The average value of roundness for each sieve fraction was calculated and the value for the sand was obtained as the weighted average of the values for each fraction.

Surface texture includes those aspects of shape whose scale is so small that they do not appear to affect the overall shape of the particle. Descriptions of surface texture usually include two groups of characteristics. The first set consists of the dullness or polish of the grain surface. In the second category are the various surface markings in the form of striations and scratches, percussion marks, surface indentations and pits etc. Surface textures are usually studied using optical, electron and scanning electron microscopes. A quantitative evaluation of surface roughness is seldom attempted and usually only a qualitative description is given.

Other Shape Indices and Shape Types

It can be shown that sphericity and roundness alone are not sufficient to uniquely define the shape even for relatively simple cases. Therefore in the case of sedimentary particles with a variety of complex shapes the problems are far more severe. Hence several other indices have been proposed and used by sedimentologists to characterize other aspects of shape. An important contribution was made by Zingg (1935) who proposed the use of two shape indices d_I/d_L and d_S/d_I, where d_L, d_I, d_S represent the longest, intermediate and the shortest particle dimensions respectively. These indices are used to construct a diagram commonly called the Zingg diagram, which is divided into four quadrants defining four main shape classes — tabular (oblate spheroid or disc shape), equant, bladed and prolate spheroid (rod or roller shaped). Another commonly used index is the shape factor (SF) for which many definitions have been proposed. A popular definition is given by

$$\mathrm{SF} = \frac{d_S}{\sqrt{(d_L/d_I)}} \tag{2.1}$$

Application Of Image Analysis

A major reason for the cursory attention paid to the morphological aspects of sands by geotechnical engineers could be the paucity of efficient and reliable methods for characterization of morphology. The conventional methods used by sedimentologists are in general time consuming and often lack objectivity. Rahim (1989) developed systematic procedures for the measurement of various shape characteristics like shape factor, sphericity and roundness. All measurements were done using an image analyzer. The general procedure for sampling and averaging was the one proposed by Youd (1973). Sphericity and shape factor were calculated based on measurements of d_L, d_I and d_S on the projected images of the grains. For the measurement of roundness an independent method was developed. Using the image analyzer, a count of the total number of corners or protrusions on the boundary of the projected image of the particle was obtained. The image analyzer provides a total tangent count for both convex and concave features on the grain boundary. This tangent count was taken as a measure of grain angularity.

Methods Based on Bulk Properties

Direct determination of shape entails measurements on individual particles which can be often cumbersome, time consuming and expensive. Therefore efforts have been made to relate morphological characteristics of the particles with some of the bulk properties of the granular material. Beddow (1980) lists some of the properties with which correlations of particle shapes have been attempted. These include flow rates, permeability, porosity and bouncing. It is clear that since such properties would be influenced by characteristics other than shape, these methods may not be as reliable as direct measurement. However considering the simplicity of some of these techniques, they may be of considerable value as tools for obtaining an approximate estimate of shape characteristics.

Mathematical Techniques

Progress in the area of image processing technology has led to the development of highly sophisticated mathematical techniques for representation of particle shapes and also for generation of these shapes mathematically. Beddow (1980) describes many such methods. The first step in these methods is to measure a large number of sample points on the particle surface and to collect all information regarding particle morphology. The next step is to condense and manipulate this information in various ways in order to develop sets of significant features, such that these features represent the original pattern as accurately as possible. Shape is represented by sets of appropriate coefficients handled in various ways. Beddow has listed four categories of methods—matrix mapping, polynomial generation, Fourier series analysis and syntactic methods. While in terms of accuracy and flexibility these methods may be the most advanced, it is doubtful whether such methods would ever become popular with geotechnical engineers. Considering the great variability in particle shapes and the considerable empiricism involved in geotechnical analysis and design procedures, it cannot be expected that such complex and rigorous methods would be employed for routine characterization of particle shapes.

2.1.2 Morphology and Fabric

The behaviour of granular materials may be intuitively expected to be strongly influenced by the geometrical, physical and mechanical characteristics of the constituent grains, their arrangement (fabric) and mutual interactions. While a great deal of attention has been given in the past to the fabric of clays, it is only over the past two decades that similar attempts are being made to understand the nature of the fabric of sands and its relationships to behaviour. Prior to these developments the only quantitative measure of fabric available was the void ratio (e) or porosity which expresses the state of packing. It was soon realized that in comparing sands with different grain characteristics, void ratio as such did not correlate well with behaviour. It was thought

that these differences could be taken into account by normalising the void ratios with respect to the maximum and minimum void ratios. Thus relative density (I_D) was proposed as a better indicator of the state of sands. Even this did not succeed in resolving the problem completely.

It is well-known that two different sands at the same relative density may show large differences in response. Many researchers have also shown that specimens of a sand having the same void ratio, but prepared by different methods, may exhibit significant variations in behaviour. It has also been demonstrated that sands are often strongly anisotropic in their strength and deformation properties. Some of these complexities in behaviour have been attributed to the differences in fabric. Hence it is clear that fabric is an extremely important factor governing the behaviour of sands and a rational understanding of this aspect is necessary for further progress in understanding and predicting the response of sands. Therefore it is worthwhile to examine the relationships between the morphological characteristics and fabrics of sands. But before this some elaboration of the concept of fabric and its characterization is necessary.

Concept Of Fabric

Brewer (1964) defined fabric as the spatial arrangement of solid particles and the associated voids. Mitchell (1993) defined fabric as the arrangement of particles, particle groups and pore spaces in a soil. According to Oda (1972a, 1972b, 1977) and Oda et al. (1980) description of fabric should consist of the following important aspects.

1. A measure of the orientation of individual particles (orientation fabric)

2. A measure which reflects the mutual relationship of individual particles (packing). As the granular materials derive their strength and stiffness from the resistance developed at the inter-particle contacts, it may be intuitively expected that the number of contacts and their distribution may be the most important aspects of packing. Oda et al. (1980) considers the following three factors to be the most significant.

- Coordination number: It is the number of contacts a particle makes with its adjacent particles.
- Standard deviation of the coordination number: In addition to the mean value of the coordination number, its dispersion represented by the standard deviation, may also be important.
- Orientation of contact normals which may be quantified in terms of the probability distribution function of the angles made by the normals to the tangent plane passing through the inter-particle contacts, with a set of reference axes.

The relationship between morphology and fabric can be examined at a fundamental level by investigating the influence of morphology on each of these four principal elements characterizing the fabric of sands. However it is suggested that instead of coordination number void ratio be considered. The practice of describing the state of packing in terms of void ratio is well established and there is considerable information available on this aspect. As it has been shown that there is a unique relationship between void ratio and coordination number (Grivas and Harr 1980) the fundamental nature of these considerations are not compromised in any way. Further, the role of particle morphology is better appreciated by considering its influence on the range of possible void ratios which is indicated by the limiting values of the void ratio e_{max} and e_{min}. Therefore the information available in the literature regarding the relationships between fabric and morphology is reviewed in terms of the effect of particle shape on the four aspects of fabric considered to be the most important. These are the maximum and minimum void ratios, the orientation of the particles, the orientation of the contact normals and the standard deviation of the void ratio.

Limiting Void Ratios

The significance of e_{max} and e_{min} has been well understood in connection with the importance of describing the behaviour of sands in terms of their relative densities. Therefore a considerable amount of research effort has

been directed towards elucidating the factors controlling the maximum and minimum void ratios. (Kolbuszewsky 1948, Alyanak 1961, Dickin 1973, Youd 1973 and De Jaeger 1991, 1994). The results of these investigations conclusively demonstrate that the most important factors affecting e_{max} and e_{min} are the particle shape and size distribution. These investigators showed that as roundness increase both e_{max} and e_{min} decrease, while De Jaeger also showed e_{max} and e_{min} decreasing with increasing sphericity also.

There appears to be two extreme points of view regarding the potential application of the relationship between morphological characteristics and limiting void ratios. Youd (1973) expressed dissatisfaction with the commonly used procedures for determining e_{max} and e_{min} and suggested that a better technique may be to estimate them from the relationships he has developed, which give e_{max} and e_{min} as a function of R and coefficient of uniformity (C_u). On the other hand De Jaeger (1994) argued that characterization of grain shape based on measurements performed on individual grains is slow, difficult and expensive and suggested that e_{max} may be used as a shape index, after the effect of C_u has been accounted for.

In view of the difficulties associated with direct determination of particle morphology, it is doubtful whether Youd's suggestion would gain wider acceptance. Definitely, the problems enumerated by Youd should be resolved by devising better methods for the determination of e_{max} and e_{min}. But in view of the enormous influence of morphology on the behaviour of sands and the need for simple methods for the characterization of morphology, De Jaeger's proposal regarding the possibility of utilizing e_{max} as an index of particle shape needs to be examined carefully.

Youd's data showed excellent correlation of e_{max} with R and C_u. On the other hand De Jaeger (1994) plotted e_{max} of a large number of sands varying by their nature, origin, grain shape and size but sieved to obtain the same C_u, against R and found that there was considerable scatter, even though the over all trend was similar to that observed by Youd. The scatter may be partly due to errors in measurement and partly due to the fact that R and C_u may not be sufficient to account for the influence of shape and

size distribution completely. In spite of these complications, a relationship linking morphology and e_{max} may be of great practical value, although this may give only an approximate estimate of shape. Therefore further efforts in this direction would be of considerable interest.

Orientation Fabric

With the exception of mica, most non-clay minerals in soils occur as bulky, but rarely as equi-dimensional particles (Mitchell 1993). Both in the laboratory and in the field these grains are deposited under the influence of a gravitational force field and in addition may be subjected to forces arising from air and water currents, vibrations, tapping, impact etc. The final orientations attained by the particles at their equilibrium positions would depend on their shape and the forces acting on them during deposition.

Oda (1972a,b) and Mitchell (1993) have presented evidence to show that there may be a preferred orientation of the particles as given by the probability distribution function of the inclinations of the long axes of the particles with a set of reference axes. The nature and degree of this preferred orientation are mostly determined by the shape characteristics of the grains and the method of deposition. If the particles are truly spherical, then there cannot be any preferred orientation. But as the departure from a truly spherical shape increases, the potential for preferred orientation also increases. In situations where gravitational forces are predominant, like pluviation in air or water, it has been observed by Oda (1972a) and Mitchell (1993) that there is a strong preferred orientation of the long axes of the particles in the horizontal direction. Arthur and Menzies (1972) have reviewed the work of several researchers and arrived at similar conclusions. Orientation fabric has some influence on the engineering behaviour of sands. It appears that particle sphericity is a major factor controlling the orientation fabric.

Orientation of Contact Normals

Inter-particle contact orientations can be described in terms of the probability distribution function of the angles between a vector which is perpendicular

to the tangent plane passing through the point of contact (contact normal) and a set of reference axes. Oda (1972a) presented a systematic procedure for this by examining thin sections cut from the samples in different directions. Many investigators have shown that the distribution of contact normals is rarely isotropic and there may be a preferred orientation of contact normals depending on the method of deposition and that this may have important implications on many aspects of soil behaviour. Therefore it is worth examining whether morphology has any influence on the distribution of contact normals.

Kallstenius and Bergau (1961) studied the fabrics of assemblages of glass spheres deposited by pluviation in air and found that the number of contacts in the vertical and horizontal planes to be different. Oda (1972a) investigated the fabrics of four sands having different shape characteristics and found that all sands showed a high concentration of contact normals in the vertical direction. While in terms of orientation of the long axes of particles there were significant differences which correlated well with shape characteristics. such a distinction could not be made in the case of contact normals. The work of Mulilis et al. (1977) indicates that the method of deposition may be the most important factor controlling the orientation of contact normals. The limited information available seems to suggest that particle morphology may not have a pronounced effect on the directional distribution of contact normals which appears to be mostly controlled by the method of deposition.

Frequency Distribution of Local Void Ratio

Several investigators have suggested that the behaviour of a sand is determined not only by the mean void ratio or coordination number but also by the deviation of these quantities about the mean. Bhatia and Soliman (1990) cite the work of Mogami (1965) which show that the angle of shearing resistance is related to both the mean and standard deviation of the void ratio. Mogami arrived at this conclusion by theoretical reasoning based on thermodynamical considerations and a probabilistic approach.

Oda (1977) conducted experiments on assemblies of glass spheres and

found that the angle of shearing resistance depends on both the mean and standard deviation of the coordination number. Bhatia and Soliman (1990) made an attempt to correlate the angle of shearing resistance (ϕ) of a variety of granular materials with the standard deviation of void ratio, but in view of other differences among the materials used by them that may influence ϕ, an accurate assessment of the effect of the standard deviation of the void ratio is difficult in this case. Kuo and Frost (1995) have reviewed the work of Ibrahim and Kagawa (1991) who investigated the effect of specimen preparation method on the fabric and cyclic behaviour of sands and observed that at the same void ratio, different methods result in different distributions of local void ratios and suggested that these could partly account for the variations in behaviour.

Bhatia and Soliman (1990) have examined the effect of particle shape characteristics on the frequency distribution of the local void ratios. They prepared samples of glass spheres, a rounded Ottawa sand and an angular crystal silica sand at comparable relative densities. Thin sections were cut from these samples and the frequency distribution of the local void ratio was determined using an image analyzer. They found that at a given relative density glass spheres had the least and the angular sand had the maximum standard deviation of the void ratio.

Recently Kuo and Frost (1995) have developed a method for quantifying the local void ratio distribution of granular materials and have presented data for a subrounded Ottawa sand. They observed that the mean of the local void ratios on horizontal and vertical planes did not show a significant variation and commented that since the grains were subrounded a large difference was not expected. On the other hand they found the deviations of void ratio to be more pronounced on horizontal planes and suggested that this could be one of the factors leading to anisotropy in fabric. It appears that these differences could be more pronounced in the case of angular sands.

Morphology, Fabric and Anisotropic Behaviour

At present most of the commonly used procedures for predicting the strength and deformation behaviour of sands assume the behaviour to be isotropic. But there is large body of work which show that the response of sands are often strongly anisotropic. (Oda 1972a,b, Arthur and Menzies 1972, Arthur et al. 1977, Oda et al. 1978, Arthur et al. 1980, Oda 1981, Symes et al. 1984, Wong and Arthur 1985, Miura et al. 1986, Chen et al. 1988. Ishibashi et al. 1991). The nature and types of anisotropy and their relevance to problems in geotechnical engineering are reviewed by Ladd et al. (1977). Jamiolkowski et al. (1985) and Mitchell (1993).

Jamiolkowski et al. (1985) classified the different types of anisotropy as:

- Inherent anisotropy—caused by the anisotropy in initial fabric.
- Initial shear stress anisotropy— due to the anisotropy in initial state of stress.
- Initial strain induced anisotropy—due to pre-straining. pre-shearing etc.
- Evolving anisotropy—which refers to the changes occurring in the initial anisotropy during subsequent application of stress or strain.

In the present context, the influence of particle morphology on anisotropic behaviour is of interest. On the basis of the preceding discussion of the nature of fabric, inherent anisotropy may be considered to be caused by the following factors.

- Preferred orientation of the long axes of the particles.
- Preferred orientation of contact normals.
- Directional differences in the distribution of the local void ratio.

It is clear that particles of low sphericity would have a preferred orientation of long axes. Also it may be expected that sands of low sphericity

and roundness may exhibit larger variations in the distribution of local void ratios. The effect of shape on the distribution of contact normals appears to be not very important. Thus while most sands irrespective of particle shape could be expected to possess some form of inherent anisotropy, sands composed of particles with low sphericity and high angularity are likely to show more pronounced inherent anisotropy.

Oda (1972a) in his studies on the effect of the initial fabric on the strength and deformation characteristics of two sands, one consisting of nearly spherical grains and the other of elongated and flat grains, showed that the anisotropic nature of deformation modulus was similar in both cases. He concluded that the deformation response was mostly determined by the distribution of contact normals and particle orientation by itself did not have a significant influence. On the other hand he observed that while mobilised strengths of the sand having high sphericity was independent of initial fabrics. the sand with low sphericity showed a clear anisotropy in terms of strength. The explanation given by Oda is that, because of the relatively larger mobility of the spherical particles, rearrangements and reorientations could occur freely in the case of sands composed of such particles and hence the effects of initial anisotropy are to a large extent obliterated in the course of straining. But in the case of sands composed of particles of low sphericity, there would be considerable resistance to rearrangement and reorientations and therefore the effects of initial anisotropy are manifested even at large strains and will be reflected in the mobilised strengths.

Effect of Method of Deposition

There is ample evidence to show that specimens of a sand prepared at the same void ratio by different methods may exhibit considerable differences in behaviour. (Oda 1972a,b, Ladd 1974, 1977, Mulilis et al. 1977, Tatsuoka et al. 1986, Ibrahim and Kagawa 1991). While Oda's objective was to examine the nature of fabric and its relationships to behaviour, the other investigators were primarily interested in the use of laboratory tests to evaluate the liquefaction potential of cohesionless soils. These workers have attributed such

variations in behaviour to differences in fabrics of the specimens prepared by various methods. The important factors identified by them include:

- Macroscopic non-homogeneities like layering and changes in relative density in different parts of the specimen (Ladd 1974, 1977, Mulilis et al. 1977).
- Differences in grain and inter-particle contact orientations (Oda 1972,a,b. Ladd 1977, Mulilis et al. 1977).
- Differences in distribution of local void ratio (Ibrahim and Kagawa 1991).
- Segregation of particles (Ladd 1977).

Based on the results of these studies it is difficult to make a quantitative assessment of the influence of morphology on the effect of method of specimen preparation as these did not involve a systematic examination of the effects of morphology. However recalling the previous discussions on the relationship between morphology and fabric it may be expected that sands of low sphericity and roundness are likely to be more sensitive to the method of specimen preparation. This is because these sands may have a stronger preferred orientation of particles and may exhibit larger variations in the distribution of local void ratio.

The work of Phool Chand (1987) appears to support the view that angular sands are more susceptible to the effects of method of deposition. He determined the angles of shearing resistance of three sands by conducting direct shear tests on specimens prepared at the same relative density, but by two different methods, namely pluviation and vibration. He observed that for a wellrounded sand the method of specimen preparation did not significantly influence the behaviour. But in the case of two angular sands vibrated specimens exhibited a larger values of ϕ than the pluviated ones.

The results obtained by Shahu (1988) also indicate that particle shape is an important factor controlling the sensitivity of the behaviour of sands to the effects of method of specimen preparation. He conducted model plate

load tests on a variety of sands deposited by different methods. The required relative density was achieved by pluviation in air, by vibration or by tapping the sides of the container. He observed differences in the load settlement characteristics in terms of the failure stress and sometimes even in the type of failure, for a sand at comparable relative densities, but prepared by different methods. The data obtained showed that particle shape may be an important factor controlling the extent to which the response is dependent on the method of deposition. Standard sand with the highest sphericity and roundness showed the least difference while Calcareous sand which consisted of sea shells of very irregular shape had the maximum effect. For Kalpi and Ganga sands which were much more angular in comparison to Calcareous sand, the effect of method of deposition was somewhat less than that for the calcareous sand. These results seem to imply that more than roundness, the overall shape as indicated by the sphericity may be the most important morphological characteristic influencing this type of behaviour.

2.1.3 Morphology and Strength Behaviour

An important task of geotechnical engineers is to ensure the safety of structures and constructions against collapse. For this an accurate assessment of shear strength of soil is essential. Therefore it is relevant to explore the relationship between morphology and strength of sands. In this subsection the available information on the effects of morphology on various aspects of strength behaviour like angle of shearing resistance, penetration resistance and liquefaction resistance is reviewed.

Angle of Shearing resistance

It is well-known that the angle of shearing resistance (ϕ) of a sand is not a fundamental property, but depends on many factors like the mineralogy, morphology and size distribution of the constituent grains, fabric, stress history, current state of stress and the applied stress path. This review is only concerned with the effect of morphology. It is suggested that effects of mor-

phology is appreciated better by examining its influence on the components of the angle of shearing resistance.

Rowe (1962) proposed that the angle of shearing resistance can be represented as the sum of three components, expressing the contributions of the sliding resistance at contacts, particle rearrangements and dilation. But in view of the difficulties involved in the determination of the first two components individually, a more pragmatic approach has been to separate ϕ into a constant volume friction angle (ϕ_{cv}) corresponding to the critical state and a component due to dilatancy. Therefore in the following the influence of morphology on ϕ_{cv} and the dilatational component is considered.

Constant Volume Friction Angle

There is ample amount of information in the literature testifying to the marked influence of particle morphology on the constant volume friction angle of sands.

Sowers and Sowers (1951) while giving typical values of angle of shearing resistance for loose uniformly graded sands recommend a value of 30° for rounded sands and 35° for angular sands. Similarly Terzaghi and Peck (1967) suggest a value of 27.5° as representative of a loose uniformly graded rounded sand and a value of 33° for a loose well-graded angular sand. Frederick (1961) in his studies on the influence of particle shape on the properties of sands obtained the angles of shearing resistance of two sands at different relative densities. The data indicates that in the loose state the sand with lower sphericity and roundness had a higher value of ϕ. Castro (1969) determined the effective angles of shearing resistance of three sands in the loose state. He reported values of 38° for a very angular sand, 35° for an angular sand and 30° for a sand with subrounded to subangular grains. As the angle of shearing resistance in the loose state may be expected to be close to the constant volume friction angle, these values may be assumed to be a good approximation of ϕ_{cv}.

Koerner (1970) investigated the effect of particle shape on the strength characteristics of three artificially prepared quartz sands. These sands had

the same mean particle size and gradation, but differed in sphericity and roundness. He observed that the *frictional* component of ϕ (which was obtained by subtracting the dilatational component from the peak friction angle using a stress dilatancy theory) increases as the sphericity and roundness decrease. As in these tests all other factors likely to influence ϕ_{cv} were controlled his results convincingly demonstrate the effect of particle morphology on ϕ_{cv}. Holubec and D'Appolonia (1973) examined the variation of ϕ with relative density for a subrounded sand and two angular sands. The data obtained by them clearly indicates that ϕ_{cv} values of angular sands are greater than that of the subrounded sands by about 4 to 5°. Lambe and Whitman (1979) state that even when a sand is strained to its ultimate condition, so that no further volume change is taking place, sands consisting of angular particles have larger angles of shearing resistance.

Negussey et al. (1988) carried out a detailed study on the constant volume friction angles of granular materials, using ring shear tests. They obtained a value of about 30° for a rounded Ottawa sand which consisted entirely of quartz and for two angular mine tailings with about 65% feldspar and 35% quartz a value of approximately 35°. While differences in mineralogy may have contributed to the high value of ϕ_{cv} for the angular sands, it is plausible that morphology also had a major role. Been et al. (1987) present data on the steady state friction angles of a variety of sands which consisted predominantly of quartz and had coefficients of uniformity between 1.3 and 2.0. The values ranged from 28.5° for a rounded sand to 35° for an angular sand, with subrounded and subangular sands having intermediate values. These results clearly demonstrate the effect of particle shape. It may also be noted that Negussey et al. (1988) have shown that steady state and constant volume friction angles are identical for all practical purposes.

A summary of the available data on the ϕ_{cv} for a large number of sands was presented by Stroud (1989). An examination of this data shows that the most important factors controlling ϕ_{cv} are particle shape, gradation and mineralogy. For uniformly graded quartz sands the values range from about 30° for rounded sands to about 36° for very angular sands. Stroud plotted ϕ_{cv}

against R and observed a well defined relationship in the case of uniformly graded sands. Pandey (1993) also found that ϕ_{cv} values obtained in direct shear tests for three sands showed a strong correlation with roundness, with ϕ_{cv} decreasing as R increases.

Thus the information available in literature convincingly demonstrates the significant effect of particle morphology on the constant volume friction angle of sands. Changes in angularity alone can lead to changes in the value of ϕ_{cv} by as much as 6°.

Dilatational Component

One of the most remarkable features of granular materials is that during a change in the state of stress, volumetric strains are induced by the deviatoric component of the stress tensor also. It is well-known that *dense* sands dilate and *loose* sands contract during shear. Thus in addition to the frictional resistance developed at the points of contact, the work involved in effecting volume changes also contribute to the shear strength of these materials. In fact the entire change in ϕ due to changes in relative density can be ascribed to the dilatational component. In view of the profound influence of dilatancy on the angle of shearing resistance (and also on other properties) of sands, the relationship between morphology and dilatancy characteristics would be of considerable interest.

While there seems to be a broad consensus on the relationship between morphology and ϕ_{cv}, there appears to be some confusion regarding the effects of morphology on dilatancy. This may be because, unlike ϕ_{cv} which is believed to be a constant material property (Negussey et al. 1988), dilatancy characteristics of a sand are strongly influenced by its state, especially relative density and confining stress. Therefore for a proper appreciation of the effects of morphology, comparisons should be made in terms of the dilatational component at the same relative density or the nature of the variation of ϕ with relative density, at comparable stress levels. Accordingly the experimental data available in literature is reviewed. Also some empirical methods in which the dilatational component of ϕ is related to a state variable which

combines the influence of void ratio and stress level, are examined with regard to the effects of morphology.

Jones (1961) determined the angle of shearing resistance of two sands by conducting direct shear tests. He found that at the same relative density, the sand with lower sphericity and roundness had a larger dilatational component. However he did not specify the stress level at which these tests were conducted. Frederick (1961) carried out direct shear tests on three granular materials at various relative densities and with normal stress of 30, 80 and 140 kPa. The materials tested were Stone Court sand which had the lowest values of sphericity and roundness, Leightonbuzzard sand with intermediate values and glass beads having the highest values, with similar particle size and gradation. Though there was considerable scatter in the data. the average value of dilatational component was found to be the highest in the case of glass beads and the lowest for Stone Court sand, at the same relative density. This appears to indicate that the tendency to dilate increase as sphericity and roundness increase. It is interesting to note that the two sands used by Jones were the same as those used by Frederick, but the results obtained by them appear to be contradictory.

Koerner (1970) has presented data on the dilatational component of the shearing resistance of three artificially prepared quartz sands having the same effective size and gradation but different morphological characteristics. All sands were tested in drained triaxial compression at a cell pressure of 207 kPa. Koerner in his paper made the observation that the dilatational component of strength was approximately the same for all three sands. He then concluded that dilatancy contribution was independent of particle shape and the entire effect of particle shape on the angle of shearing resistance was on the frictional component. However Giroud (1971) in a discussion on Koerner's paper pointed out some errors in the plotting of data. He argued that Koerner's attempt to represent the variation of the dilatational component of ϕ with relative density for all three sands with a single curve did not do justice to the data presented. In the light of Giroud's critical comments, a close examination of Koerner's data shows that the behaviour of the three

sands are indeed distinct. At the same relative density, the subrounded sand has the largest dilatational component, the subangular sand intermediate values and the angular sand has the least. It may also be seen that the increase in dilatancy due to increase in relative density is least for the angular sand and maximum for the subrounded sand.

Billam (1971) investigated the effect of confining stress on the dilatancy characteristics of a variety of cohesionless geological materials. He found that when the confining stress was normalized with respect to the tensile strength of particles, materials composed of rough grains had higher rates of dilatancy at lower stress levels compared to smooth grained materials. Holubec and D'Appolonia (1973) examined the influence of particle shape on the variation of the angle of shearing resistance with relative density by conducting drained triaxial tests at a cell pressure of about 190 kPa. The materials tested were glass beads, a subrounded sand and two angular sands. The particle size distribution of these materials did not differ appreciably. He observed that the glass beads exhibited the least increase in ϕ with relative density and the subrounded sand the maximum, while angular sands showed intermediate increase. It may be noted that while the behaviour of the three sands show a consistent relationship with particle shape, the behaviour of glass beads appears to be somewhat anomalous, the reasons for which are not clear.

A review of the data available in literature on the strength and dilatancy of seventeen sands is presented by Bolton (1986). The angles of shearing resistance were obtained from triaxial and plane strain tests with mean effective normal stress at failure ranging from 150 to 600 kPa. He found that the contribution due to dilatancy did not appear to depend on the type of sand and suggested a unique relationship between the dilatational component of ϕ and relative density which was deemed to be applicable in this stress range.

Phool Chand (1987) conducted direct shear tests on a rounded sand and two angular sands at relative densities of 25% and 55% with normal stress ranging from 50 to 1600 kPa. Based on the results of these tests he derived expressions for the shear strength as a function of relative density and normal stress and used them to predict the variation of shear strength with relative

density in the normal stress range of 50 to 1000 kPa. He observed that the gain in strength with relative density was more for the rounded sand. Thus his results indicate that the dilatational component is larger in the case of rounded sands compared to angular sands, in this range of stresses. However it may be noted that these conclusions are based on results obtained for a relatively narrow range of relative densities.

A detailed investigation of the effects of particle shape on the strength of sands was carried out by De Jaeger (1994). He arrived at the conclusion that at a given relative density, the shear strength is higher for angular sands than for rounded ones, and this difference decreases as the mean normal stress level increases. This is equivalent to saying that at low stress levels angular sands possess a larger dilatational component than rounded sands and as stress levels increase the tendency for angular sands to dilate more is increasingly suppressed. De Jaeger explained this by postulating that, at low stress levels the predominant mechanism of grain movement is rolling which is sensitive to particle shape, whereas at higher stress levels sliding, grain deformation and crushing which are relatively less sensitive to shape are the dominant mechanisms.

This review has brought to focus some of the conflicting points of view advanced by different workers regarding the relationship between morphology and dilatancy. Some have found that angular sands dilate more than rounded sands at the same relative densities, while there are others who hold that the opposite is true. There is also a third school of thought which considers the dilatational component of ϕ to be independent of particle shape. It is quite possible that all these points of view may be correct depending on the stress levels at which one is operating. Thus some of these apparent contradictions may be resolved by considering the effect of stress level on dilatancy.

The most important factors controlling the dilatational component of ϕ of a sand are relative density and the magnitude of normal stress. Some empirical methods have been developed where the dilatancy characteristics are correlated to a state variable combining the influence of void ratio and stress level. In view of the effects of particle shape on dilatancy it would be

of some interest to examine whether this is reflected in these methods. In the following two such methods will be reviewed. These are Bolton's relative dilatancy index (Bolton 1986) and the state parameter approach of Been and Jefferies (1985).

Relative Dilatancy Index

Based on the data obtained in plane strain and triaxial tests on 17 sands Bolton proposed a relative dilatancy index I_R given by

$$I_R = I_D(10 - \ln P'_f) - 1 \tag{2.2}$$

where P'_f = mean effective normal stress at failure in kPa. He then related the dilatational component of the angle of shearing resistance to the relative dilatancy index through the following relationships.

$$\phi' - \phi_{cv} = 5I_R \quad \text{for plane strain}$$
$$\phi' - \phi_{cv} = 3I_R \quad \text{for triaxial}$$

Bolton observed that for the data considered by him for which P'_f was in the range of 150 to 600 kPa these expressions gave a good fit irrespective of the type of sand. He concluded that these expressions were valid in the range of $0 < I_R < 4$.

Bolton also recognized the fact that particle crushing would reduce the critical mean effective stress at which dilatancy was suppressed and to account for this gave the following generalized definition of the relative dilatancy index.

$$I_R = I_D(Q - \ln P'_f) - R^\star \tag{2.3}$$

where Q and $R^\star$ are empirical constants. He also suggested some typical values of Q depending on the mineralogy. Bolton appears to relate crushing solely to the mineralogy whereas it is well-known that particle shape also plays an important role in crushing. An angular sand will start crushing at a lower stress compared to a rounded sand and hence is likely to have a

lower value of Q. Therefore Bolton's method would always predict a lower dilatational component for angular sands than for rounded sands. But in principle it is possible to account for the effect of particle shape by choosing appropriate values of Q and R^*.

State Parameter

In the course of their research directed towards the development of rational procedures for the engineering of hydraulic sand-fills Been and Jefferies (1985) came to the conclusion that to satisfactorily correlate the engineering properties of a sand to its state, a state variable combining the influence of the state of packing and stress is required. They proposed void ratio and mean normal effective stress as the appropriate measures and the steady state as the reference state. They defined a new state variable given by the difference between the current void ratio and the void ratio on the steady state line at the current mean normal effective stress and called this as the state parameter (ψ). Good correlation was observed between the state parameter and many engineering properties. The purpose of this review is to examine whether these correlations are valid for all types of sands. irrespective of their morphology.

If the effects of particle shape are to manifest themselves in the state parameter. a necessary condition is that the same should be reflected in the reference state. Been and Jefferies (1985) found that the slope of the steady state line increases with increasing fines content, which according to them was consistent with a trend towards greater compressibility with increasing fines content. As a logical corollary, it may be assumed that angular sands being more compressible, should have steeper slopes. This has been convincingly demonstrated by Castro et al. (1982) who have presented steady state lines for subrounded, subangular and angular sands. The slopes of steady state lines were found to increase with increase in angularity of the grains. Thus it is evident that the effects of particle shape are manifested in the steady states.

The second question is whether the state parameter leads to correlations

which are applicable to all sands irrespective of differences in morphology. Been and Jefferies (1985) in their original paper related the dilatational component of the angle of shearing resistance of a number of sands to the state parameter and observed a well defined relationship despite some scatter, there by implying that these correlations were applicable to all sands. However in their reply to the discussions on this paper they stated that this may not be correct since the data were all for subrounded to subangular sands (Been and Jefferies 1986). They also presented data for an angular tailings sand which exhibited a lower dilation rate compared to the subangular and subrounded sands at the same state parameter. They commented that some form of normalisation of the state parameter was required if it is to be returned to the concept of universal applicability irrespective of sand grading, type or mineralogy.

Thus it is clear that the state parameter approach fails to provide a unique correlation between the state and dilatancy characteristics which is applicable to all sands. Therefore for a realistic portrayal of the behaviour of sands considerations of particle morphology cannot be overlooked.

Penetration Resistance

The difficulties in obtaining undisturbed samples of sands often preclude the use of laboratory tests to evaluate their engineering properties. Therefore design parameters for sands are usually determined by conducting in-situ tests, the most popular being penetration tests. Penetration tests do not themselves directly yield any basic engineering property like angle of shearing resistance or deformation modulus. These tests are also difficult to model because of the complex stress paths and boundary conditions involved and hence the possibility of back analyzing the basic material properties from the measured response is also remote. Hence the general practice is to establish empirical correlations between various mechanical characteristics and the response in penetration tests and then use these to obtain the required parameters.

It is clear that it may be difficult to establish these correlations for each

and every sand individually. But it is also true that if differences in particle characteristics affect the response, then correlations developed for a particular sand cannot be indiscriminately applied to a different type of sand. Therefore the effects of particle shape on penetration resistance are of considerable interest in the interpretation of penetration tests.

Holubec and D'Appolonia (1973) investigated the effect of particle shape on the resistance to dynamic penetration, by conducting miniature penetration tests on four different types of granular materials. They found that at a given relative density angular sands had a higher penetration resistance than rounded sands. In the case of angular sands the increase in penetration resistance with relative density was much more pronounced than for rounded sand. However it should be noted that these tests were performed at extremely low levels of confining stress.

Marcuson and Bieganousky (1977a,1977b) conducted standard penetration tests in a large chamber at different stress levels on four sands. They observed that a relationship correlating N values from the standard penetration tests with relative density and overburden stress was not universally valid for all sands. As the grain shapes of these sands were different it is plausible that this could be in part due to these differences.

Robertson and Campanella (1983) stated that a unique relationship between cone resistance, in-situ effective stress and relative density cannot be expected since other factors such as compressibility also influence cone resistance. They have also compared such relationships obtained by different researchers and shown that other factors being same compressible sands had a lower cone resistance. It may be noted that compressibility is related to angularity.

Been et al. (1987) have analyzed the relationship between cone tip resistance and sand state based on published results of calibration chamber tests on six sands with shapes ranging from angular to rounded. They have related the normalised cone tip resistance $(q_c - P)/P'$ where P is the mean normal stress, to the state parameter ψ. These relationships were shown to be significantly influenced by particle angularity. At a given value of ψ rounded sands had the maximum normalized cone resistance and angular sands the

least. The trend of the variation of normalized cone resistance with the state parameter was found to be similar for all sands.

The effect of particle characteristics on the penetration resistance of sands, under conditions of very low levels of confining stress was investigated by Shahu (1988). He conducted model penetration tests on four sands at various relative densities. He found that at a given relative density penetration resistance was highest for Kalpi and Calcareous sands, followed by Standard sand and the least for Ganga sand. Kalpi and Ganga sands are angular sands while Standard sand is wellrounded and calcareous sand is subrounded. The behaviour of Ganga sand which in terms of particle angularity and gradation is similar to Kalpi sand was found to be drastically different. This apparent anomaly was explained by the relatively high mica content of Ganga sand which while giving high angularity did not lead to significant interlocking.

Many other researchers have drawn attention to the influence of particle morphology on penetration resistance and the errors that could arise from using correlations developed for a particular sand for a different type of sand (Joustra and De Gijt 1982, Clayton et al. 1985, Bellotti et al. 1985, Jamiolkowski et al. 1988 and Sladen 1989).

Liquefaction Resistance

The term liquefaction has been defined as the phenomenon wherein a mass of soil looses a large percentage of its shearing resistance when subjected to undrained monotonic, cyclic or shock loading and flows like a liquid until the shear stresses acting on the soil mass are as low as the reduced shearing resistance. The possibility of failure due to liquefaction induced by earthquakes or other disturbances, is an important concern which needs to be addressed in the evaluation of the stability of earth dams, tailings dams, slopes and also structures founded on saturated sands. Research on the factors controlling resistance to liquefaction show that morphological characteristics especially the angularity of the grains have a significant influence.

Castro (1969) based on the results of laboratory investigations on the

behaviour of three sands with subrounded to subangular, angular and very angular grains, concluded that the subangular to subrounded sand is more likely to be involved in a liquefaction failure than angular and very angular sands.

The development of the steady state concepts by Castro (1969), Cassagrande (1975), Castro (1975), Castro and Poulos (1977), Poulos (1981), Castro et al. (1982) and Poulos et al. (1985) has led to improved understanding of the factors governing susceptibility of sands to liquefaction. This approach is based on the notion of the steady state of deformation which is defined by Poulos (1981) as that state in which a mass of particles is continuously deforming at constant volume, constant normal effective stress, constant shear stress and constant velocity. The conditions at steady state is usually represented by the steady state line which is a plot of void ratio against the effective minor principal stress or the mean effective principal stress. The influence of particle angularity on the steady state lines has been convincingly demonstrated by Castro and Poulos (1977), Castro et al. (1982) and Poulos et al. (1985).

Based on extensive investigations on the behaviour of a number of sands, Castro et al. (1982) summarized the effect of particle angularity as follows. As the angularity of the particles change from subrounded to angular, the shape of the steady state line (void ratio versus logarithm of effective minor principal stress) changes from nearly linear and relatively flat to non-linear and relatively steep. As grain angularity increases, the value of void ratio corresponding to $\sigma_3' = 98.1$ kPa and e_{min} increase; however, the difference between them does not change appreciably. Except for high void ratios, the more angular sands tend to have smaller differences between the peak and steady state shear strengths in comparison to the more rounded sands. The velocity of deformation at steady state is generally lower for angular sands than for rounded sands. Poulos et al. (1985) present examples of steady state lines for subrounded, subangular and angular sands. They state that the shape of steady state lines is affected mainly by grain morphology. They also recommend that the more narrowly graded and the more rounded sands

in a potential failure zone should be checked, since often these are the most susceptible to liquefaction.

Vaid et al. (1985) investigated the effect of grain angularity on liquefaction by conducting tests on a rounded Ottawa sand and an angular tailings sand. They present the important findings as follows. At low levels of confining stress ($\leq$ 200 kPa), resistance of angular tailings sand was greater than that of the rounded Ottawa sand, for the entire range of relative densities considered. Under high levels of confining stress, resistance of angular sands could be lower or greater depending on the relative density. In general for high confining stress and high relative density the rounded sand exhibited higher resistance. At low levels of confining stress, both sands showed comparable increase in resistance with relative density. But at higher stress levels the increase was larger for the rounded sand. At comparable relative densities, the decrease in resistance with increasing confining stress, was more pronounced in the case of the angular tailings.

2.1.4 Morphology and Deformation Behaviour

One of the most important requirements to be satisfied by a good design is to limit the deformations to acceptable levels. Therefore accurate predictions of deformations is a vital component of geotechnical design. For this a thorough understanding of the stress-deformation characteristics of soils is essential. In the following the existing information on the relationship between morphology and deformation behaviour of sands is reviewed. The effects of shape characteristics on compressibility, crushing, small strain shear modulus and the effect of overconsolidation on deformation characteristics are described.

Compressibility

Roberts and De Souza (1958) investigated the compressibility characteristics of sands by conducting one-dimensional compression tests. Their results indicate that angular sands are more compressible than rounded sands for stresses less than 100 MPa. De Beer (1963) reports that the compressibility

of sands increase with increasing angularity. Lee and Farhoomand (1967) employed anisotropic triaxial compression tests with constant stress ratios to study the compressibility characteristics of sands and found that angular sands have higher compressibility than rounded sands. The maximum stress applied was about 14 MPa. El-Sohby and Andrawes (1972) conducted isotropic compression tests on a variety of granular materials. They report that as the angularity of the particles increase, compressibility also increase. Here the maximum applied stress was about 620 kPa.

Holubec and D'Appolonia (1973) carried out K_0 triaxial tests on four granular materials with different shape characteristics. They found that at the same relative density, the compressibility increases with angularity. Experiments were performed with stresses up to 24 MPa. Clayton et al. (1985) have summarized the factors influencing compressibility of sands. They rank angularity along with void ratio as the most important. Yudhbir and Rahim (1987) showed that particle angularity and surface roughness increase the compressibility of sands. They have demonstrated that at comparable stress levels, effect of relative density on compressibility, is more pronounced in the case of rounded sands in comparison to angular sands. They have also shown that the increase in constrained modulus with stress at comparable values of relative density, is larger for angular sands.

Hardin (1987) developed a model for one-dimensional strain in normally consolidated sands. Using published results of investigations carried out by many researchers he has evaluated the parameters of the model. These sands ranged in shape from rounded to angular. The model parameters reflect the differences in particle shapes with angular sands being shown to be more compressible. Pestana and Whittle (1995) have present the results of one-dimensional compression tests on a subrounded sand and two angular sands for stresses up to 100 MPa. Their results show that angular sands are more compressible than the subrounded sand. They also report that the compression curve for the subrounded sand had a well defined yield point beyond which there was a large increase in compressibility. But in the case of angular sands this transition was a gradual one. The effect of particle shape

on compressibility of sands has also been studied by Kapoor (1985), Phool Chand (1987) and Rahim (1989). They have shown that compressibility is higher in the case of angular sands.

Based on this review the effects of particle morphology on compressibility may be summarized as:

- At a given relative density angular sands are more compressible than rounded sands. It may be noted that at very high stresses these differences may not be very pronounced.
- The effect of relative density on compressibility is more for rounded sands.
- The stress at which a relatively large increase in compressibility occurs is lower for angular sands.
- The transition from relatively low compressibility to high compressibility due to increase in stress level is well defined in the case of rounded sands, but not for angular sands.

Crushing of Particles

Crushing of grains rarely occur in many types of sands at the low stress levels normally encountered in common problems in geotechnical engineering. But there are many situations where the stress levels may be high enough to cause degradation of particles—high earth dams, deep mine shafts, tunnels, deep well shafts, deep driven piles; explosions and projectiles; and many geophysical applications (Vesic and Clough 1968, Yamamuro et al. 1996). Also some sands may be more susceptible to crushing at lower stress levels. Thus an understanding of the phenomena of grain crushing may be of importance in many situations, depending on the type of sand and the stress level.

Based on the work of several researchers (Lee and Farhoomand 1967, Billam 1971, Lade et al. 1996) the main effects of particle crushing on the behavior of sands may be summarized as follows.

- Large reductions in volume leading to large settlements under drained conditions and pore pressures under undrained conditions.

- Suppression of dilatancy leading to reduction in shear strength and deformation moduli, increase in failure strains and a more ductile stress-strain response.

- Lowering of the average particle size and increase in the amount of fines leading to reduction in permeability. These effects may be important in high earth dams and may lead to changes in seepage pattern and pore pressure distribution and may affect performance of gravel drains and filters.

Particle crushing can occur in many ways. Ramamurthy (1969) classified the different types of crushing as:

- Crushing or shearing of asperities on the contact surface

- Splitting of soil grains

- Breakdown of sharp asperities

- Breakdown of asperities due to stress reversal.

Particle crushing in sands is affected by a large number of factors. Many investigators have shown that particle shape is an important factor which significantly influences many aspects of crushing phenomena. Experiments performed on sands with different morphological characteristics show that, other factors being same angular sands are more susceptible to crushing than rounded sands (Lee and seed 1967, Lee and Farhoomand 1967, Vaid et al. 1985, Phool Chand 1987, Rahim 1989). Yudhbir and Rahim (1987) have shown that in the case of angular sands modification of the grain boundaries begins at stresses less than the critical stress. Here critical stress denotes the stress at which crushing and fracturing of particles becomes significant.

Hardin (1985) has analyzed the data on particle crushing for a wide variety of granular materials and developed empirical relationships to predict the

total amount of particle breakage for a given soil subjected to a specific loading. One of the factors considered by him is particle shape represented by a quantity called shape number which has been related to angularity. These relationships indicate that the amount of crushing would be more in the case of angular sands than for rounded sands. Lade et al. (1996) have discussed the role of particle angularity in increasing particle breakage. Angular particles break more easily because stresses can concentrate along the narrow dimensions, thus fracturing the particle. Stresses can also concentrate at angular contact points, causing the points to fracture.

Summarizing the available information, the important effects of particle morphology on crushing and modification of grains are:

- Under similar conditions the amount of crushing would be more for particles with low sphericity and roundness and higher surface roughness.

- Total amount of crushing consists of the contributions of the different types like splitting of grains, breakdown of sharp asperities, crushing and shearing of asperities on the contact surface etc. These phenomena would be influenced by the morphological characteristics. Hence the stress level at which these processes are initiated and the contribution from each mechanism would vary from sand to sand. In general it can be said that most of these phenomena would begin at a lower stress for angular sands.

- The size and shape of particles produced by crushing would depend on the mechanism of crushing. As particle shapes influence these mechanisms it may be expected that changes in size and shape would vary from sand to sand.

Shear Modulus

Shear modulus (G) is an important property of soils which has applications in many problems including response of machine foundations and effect of earthquakes on structures. Of late there is a growing trend towards estimation of

deformation moduli from seismic wave velocity measurements. Therefore an appreciation of the effects of morphology on the shear modulus of sands may be of considerable importance.

Hardin and Richart (1963) examined the various factors influencing the elastic wave velocities and moduli of granular materials by conducting resonant column tests. One of the variables considered by them was particle shape. They carried out tests on a rounded Ottawa sand and an extremely angular crushed quartz sand with different size and gradation characteristics. They found that at low confining stresses and at comparable void ratios the shear wave velocity is higher for angular than for rounded sands. As the confining stress increases, these differences tend to decrease. They also pointed out that grain shape affects velocity also through its effect on the range of possible void ratios. It was also observed that the presence of moisture reduced the velocity of wave propagation. This reduction was found to be maximum in the case of rounded sands whereas it was only marginal in the case of angular sands.

Hardin and Drnevich (1972) investigated the effect of various factors on the shear modulus and damping characteristics of a large variety of soils, based on the results obtained by them and also on the data available in the literature. They have assigned different degrees of importance to the various parameters. They have categorized the various grain characteristics including grain shape as relatively unimportant. Their position is that grain characteristics will affect both void ratio which is a very important parameter and the effective strength envelope which is a less important parameter. But if the void ratio and effective strength envelope of the soil are accounted for, then these parameters will also account for the effect of grain characteristics.

Krizek et al. (1974) studied the effect of particle characteristics on the propagation velocity of a rod wave. Here rod wave refers to a special form of compression wave that appears only in a rod like specimen subjected to the particular boundary conditions where lateral stresses are held constant and lateral deformations are allowed to occur. The sands tested had different shape characteristics with sphericities ranging from 0.81 to 0.87 and

roundness varying from 0.34 to 0.65. Based on the results obtained they concluded that the effects of various particle characteristics including shape were negligible, except insofar as they affect the void ratio.

Iwazaki and Tatsuoka (1977) carried out resonant column tests to study the effect of grain size and grading on the dynamic shear modulus of sands. As a part of this investigation, they conducted tests on 15 types of clean uniform sands with rounded, subangular and angular grains. They found that at a given level of strain, the dynamic shear modulus of all these sands could be satisfactorily related to the confining stress and void ratio by a single relationship. These investigators were primarily concerned with the effects of mean particle size and gradation only and did not make explicit the effect of particle shape. But the fact that a single equation could represent the shear modulus of a large number of sands having a wide range of grain shapes, indicate that the effect of shape was not really significant in this case.

Edil and Luh (1978) carried out a detailed investigation of a large number of factors including particle shape on the dynamic response of sands using resonant column tests. The sands tested consisted of more than 80% quartz and the mean sphericities ranged from 0.81 to 0.87 and mean roundness from 0.33 to 0.65. (Some of these sands appear to be the same as used by Krizek et al. 1974). They analyzed the effect of particle shape characteristics on the value of shear modulus at a strain of 0.25×10^{-4} which was termed as the reference dynamic shear modulus. It was found that in general at the same void ratio the reference dynamic shear modulus decreased as roundness increased. But when the comparisons were made at the same relative density it was found that the modulus increased as mean roundness increased, but the change in modulus with roundness was less than when compared at the same void ratio. Edil and Luh observed that by expressing the density state in terms of relative density rather than void ratio the effect of shape was normalized to some extent. They also found that the dependency of modulus on relative density was more pronounced in the case of a subangular sand than a rounded sand.

Another aspect of particle morphology considered by Edil and Luh was

the surface texture. It was found that a particular sand consistently exhibited a higher value of shear modulus than indicated by the general trend. The explanation given was that this particular sand had a somewhat smoother surface texture than the other sands. However this appears to be contradictory to the usual expectation that rough grained materials would have higher strength and stiffness.

Based on the data obtained by them Edil and Luh proposed empirical relationships for evaluating shear modulus with roundness as one of the parameters. They evaluated validity of these relationships, using the results of another series of experiments conducted by them and also the data available in literature. They concluded that the relationships proposed by them were quite satisfactory and performed better than other available relationships.

Seed et al. (1986) present data on the shear modulus for sands and gravelly soils determined by laboratory and field tests. Based on an analysis of this data, they conclude that dynamic shear modulus of granular soils is a function mainly of void ratio, grain size, shear strain and mean effective principal stress and that other characteristics have only a minor influence on the results and are usually not important for most practical purposes. Evidently they do not consider particle shape to have a significant influence on shear modulus.

Quian et al. (1993) studied the effects of particle characteristics on the small strain shear modulus of unsaturated soils. They showed that at low confining stress levels shear modulus of partially saturated sands were higher than those of dry sands because of the effects of capillary stresses. They found that particle shapes influenced significantly the behaviour of partially saturated soils. The ratio of the maximum value of shear modulus in the partially saturated condition to the shear modulus in the dry state and the optimum degree of saturation were seen to be larger in the case of angular sands compared to rounded sands having the same mean void ratio and mean particle size.

Hryciw and Thomann (1993) have developed a stress history based model for the small strain shear modulus of cohesionless soils. One of the aspects studied by them was the dependency of the shear modulus on the confining

stress. This relationship is usually represented by a power function. They state that while theories for regular assemblages of homogeneous spheres in contact, predicted a value of 1/3 for the exponent, experimental results showed a value of 1/2. They have discussed the plausible reasons for this discrepancy as pointed out by many researchers. One reason is that in real soils with non-uniform particle size distribution, the number of load transmitting contacts and the change in their number during compression, may be larger than that for a perfect array of spheres of uniform size. Another reason cited was that in soils particle contacts are not Hertzian (spherical) but point like or conical.

Hryciw and Thomann (1993) suggested that as these factors could vary from sand to sand a range of values for the exponent could be expected. They conducted tests on 7 sands with different particle sizes and gradations and having shapes ranging from rounded to angular, to study the effect of these particle characteristics on the shear modulus. The shear modulus was calculated from shear wave velocities measured in a bender element oedometer. They found that the value of the exponent in the power function relating shear modulus and confining stress was related to the value of S_{1Dmax} which is a dimensionless stiffness coefficient for 1-D strain proposed by Hardin (1987). As S_{1Dmax} increased the value of the exponent decreased. Hardin (1987) has shown that at comparable densities S_{1Dmax} increases as roundness increases. Therefore these results indicate that the value of this exponent is higher for angular sands than for rounded sands.

This review of the available information on the effect of morphology on small strain shear modulus indicate that, while there is some evidence to show that shape characteristics do have some influence, many researchers consider this to be small.

Effect of Overconsolidation on Deformation

Mechanical overconsolidation may lead to reduction in void ratios, particle crushing and increase in horizontal effective stress depending on the magnitude of stresses involved, stress-path and type of sand. These changes may

be reflected in the deformation behaviour of sand. Therefore it would be of interest to examine whether particle morphology is significant to understanding the effect of overconsolidation. The limited amount of information available on this aspect is presented here.

Many investigators have shown that the small strain shear modulus is not significantly affected by overconsolidation (Hardin and Richart 1963, Afifi and Richart 1973, Tatsuoka et al. 1979, Tatsuoka and Shibuya 1992). But there are indications that this may not valid for all sands under all conditions. There may be differences between sands regarding the effect of overconsolidation, although these may be small in comparison to the effects of confining stress and void ratio.

Hardin and Richart (1963) found that isotropically overconsolidated specimens of a rounded sand had a shear wave velocity less than that of a normally consolidated specimen at the same stress by about 1 to 4%. However in the case of an angular sand overconsolidation resulted in a slight increase in velocity.

Fioravante et al. (1994) investigated the effect of overconsolidation on the small strain shear modulus of a highly crushable carbonate sand and found that the modulus increases as the overconsolidation ratio increased. They attributed this to the increase in the number of inter-particle contacts due to the crushing of particles. Increasing the stress led to crushing and rearrangement and thus an increase in the number of inter-particle contacts. Upon unloading, since the recoverable deformation is low, the number of contacts are not changed appreciably. Hence an overconsolidated specimen would have a larger number of contacts than a normally consolidated specimen at the same stress. It may be argued that since angular sands are more susceptible to crushing than rounded sands the same considerations are applicable in this case also.

The results obtained by Phool Chand (1987) indicate that particle morphology has a significant influence on the effect of overconsolidation on the deformation characteristics. He conducted direct shear tests on a wellrounded and two angular sands at a relative density of 55% and normal stress of

100 kPa. Tests were conducted with values of overconsolidation ratio (OCR) of 1, 16 and 100. He found the effect of OCR on the initial stiffness of the wellrounded sand to be relatively small in comparison to that for angular sands. The differences were ascribed to the pronounced crushing that had occurred in the case of angular sands.

2.2 Ageing and Engineering Behaviour of Sands

The changes occurring in the state of a sand under conditions of constant overburden subsequent to its deposition, may often result in significant improvements in strength and deformation properties. The consequences of ageing after disturbance of the ground are also of interest in many situations. For a rational understanding of these phenomena, elucidation of the mechanisms of ageing is necessary. Another issue of importance is the prediction of the improvements in soil behaviour which are caused by ageing. Accordingly the available information on these aspects are examined in this section.

2.2.1 Ageing and Strength Behaviour

In this subsection the effect of ageing on the various aspects of strength behaviour of sands is reviewed. Influence of ageing on angle of shearing resistance, penetration resistance and liquefaction resistance is described. The phenomenon of sensitivity observed in some sands and thought by many researchers to be an evidence for the improvements in strength resulting from ageing is examined. The strength properties of locked sands are also discussed.

Angle of shearing resistance

Not many attempts have been made to study the effect of ageing on the angle of shearing resistance of sands in the laboratory. Daramola (1980)

conducted drained triaxial tests on isotropically consolidated specimens of a sand in which the specimens were sheared after being subjected to the confining stress for periods up to 152 days. The data presented by him show that there is no significant increase in the principal stress ratio at failure due to long term consolidation for such durations. The main effects of ageing observed in this study were a tendency for less volume decrease during shear, smaller strains to failure and increased modulus; however no increase in ϕ was indicated.

Sensitivity

The work of several researchers show that natural deposits of sands may often possess a structure which is susceptible to reduction in strength and stiffness due to disturbance. It has been suggested that this structure is a result of changes occurring with time during the post-depositional history of of the deposit.

Mitchell and Solymar (1984) present data to show that blast densification of an alluvial sand deposit initially resulted in a decrease in penetration resistance even though an increase in relative density was indicated by ground settlements. To examine this phenomenon of strength loss due to disturbance they carried out a special test. A hollow steel pipe was pushed into the undisturbed deposit and the force required for this was measured. The pipe was then withdrawn and again pushed down at the same location. It was found that the resistance to penetration had considerably reduced as a result of the disturbance. However when the same test was repeated in a layer of freshly placed hydraulic sand-fill, such an effect was not observed. Based on these observations they concluded that natural deposits of sand may have a structure that is susceptible to significant loss of strength upon disturbance.

Schmertmann (1987) in his discussion on the paper by Mitchell and Solymar (1984) noted that he had also encountered cases of an initial reduction in cone penetration resistance immediately following vibro-flotation despite a substantial increase in density. He argued that during ground densification using methods like blasting and vibro-flotation zones of very low effective

stress may be created adjacent to the zone of disturbance and this may be the reason for the immediate reductions in penetration resistance.

Hryciw and Dowding (1988) carried out cone penetration tests to evaluate changes in soil properties after blasting of a sand deposit. Tests conducted one to two days after blasting showed that tip resistance (q_c) showed a decrease from its initial pre-blast values, even though densification was indicated by ground settlements. They attributed this decrease in q_c to either a decrease in horizontal effective stress or disruption of cementitious intergranular bonds. Dilatometer tests conducted after blast revealed that the horizontal stress index had decreased after blasting.

Thomann and Hryciw (1992) present the results of investigations on stiffness and strength changes in cohesionless soils due to disturbance. They conducted some tests in a quasi-static torsional shear-resonant column apparatus in which the small strain shear modulus of laboratory prepared specimens of sands was measured, prior to and after application of high amplitude shear strain. They observed that even in these freshly deposited samples, disturbance of the structure induced by shearing strains led to significant reduction in shear modulus even though there was a small increase in density. They also report the results of a field study in which tests conducted after blasting in a natural sand deposit showed decrease in cone penetration resistance and shear wave velocity. But DMT horizontal stress index and down hole nuclear density tests suggested that no significant changes in horizontal stress and soil density had taken place. Laboratory tests on reconstituted samples of the same sand, in which the disturbance was simulated by imposing the estimated shear strains resulting from the blast, in a resonant column-torsional shear apparatus, indicated that the in-situ decrease in shear wave velocity was significantly greater than the decrease in the laboratory. It was suggested that the development of soil fabric over geologic time and its disruption could have been the reasons for this difference.

Charlie et al. (1992) carried out investigations to evaluate changes occurring due to blasting in a relatively dense, saturated alluvial deposit. Measurements of ground settlements indicated that blasting did not result in any

significant change in relative density. Cone penetration resistance conducted a week after the blast showed that tip resistance had decreased by 62% and local friction by 30%. Piezometer readings showed that pore pressures induced by blasting were dissipated in a few minutes. It was suggested that since this reduction in resistance could not have been due to changes in density or excess pore pressures disturbance of the structure might have been the reason.

All these experiences with ground modification indicate that disturbances often result in considerable reduction in strength and stiffness. But whether this reduction is caused by the disruption of a structure that has developed due to ageing needs to be examined. Changes in the state of stress, especially decrease in horizontal stress could be also a reason. Improvements in strength and stiffness may occur during the geologic history of the deposit through processes other than ageing, for example overconsolidation. Thus other causes may also contribute to sensitivity and hence a deposit of sand exhibiting sensitivity need not necessarily mean that improvements in strength had occurred due to ageing.

Penetration Resistance

Mitchell and Solymar (1984) report the results of penetration tests conducted in a 10 m thick hydraulic sand-fill. Cone penetration tests were conducted 4 to 10 days after fill placement and then repeated after 50 to 80 days. It was observed that in each case the later penetration resistance was about twice the previous value. Mitchell and Solymar (1984) have cited the work of Durante and Voronkevich (1955) who found that a freshly redeposited alluvial sand compacted to the same density as that of the natural undisturbed deposit, had a lower penetration resistance than the natural deposit. The data presented show differences of the order of 70-100%. Durante and Voronkevich attributed this higher resistance of the natural sand to a cohesion which was lost as a result of disturbance.

Mitchell and Solymar (1984) have also quoted the result obtained by Dudler et al. (1968) who carried out penetration tests on partially saturated

and clean sands, over a period extending up to 6 years. Penetration resistance increased from 3.4 blows per 10 cm after 3 days to 10.9 after 9 months for a partially saturated sand. For the same sand in the fully saturated condition the increase was from 3.3 to 4.4 blows per 10 cm. Generally the increase in strength with time was completed with in the first 2 to 3 years after deposition. They also reported that the effect of time was greater for dense and well-graded sands than for loose and uniform sands. Another source quoted by Mitchell and Solymar is Dudler and Iulin (1981) who presented data to show that the penetration resistance of a sand-fill increased by a factor of 2.4 over a period of 80 months.

Dowding and Hryciw (1986) as a part of their study of blast densification of saturated sand conducted some control tests to examine the variation of cone base resistance with time after deposition in which there was no prior blasting. The tests were performed in a large tank and the required relative density of sand was obtained by gently lifting and dropping the tank. Model cone penetration tests were carried out over a period of 15 days. It was found that tip resistance increased with time. An examination of this data shows that the increase in 15 days was about 25%.

Skempton (1986) consider ageing to be an important factor influencing the standard penetration resistance of sands. His work compares the penetration resistance measured in tests on freshly deposited specimens in the laboratory, recent sand-fills and natural sand deposits. Most of the data is from tests conducted in Japan on man-made fills and natural sand deposits. The comparison was made in terms of a normalised SPT blow count $(N_1)_{60}/I_D$ where $(N_1)_{60}$ is the measured blow count corrected to correspond to 60% free fall energy and a unit vertical effective vertical stress of 100 kPa and I_D is the relative density. He presented the following typical values for normally consolidated fine sands—35 for laboratory tests with age less than 0.1 year; 40 for recent fills of age around 10 years; and 55 for natural deposits with age greater than 100 years.

Barton et al. (1988) report the results of standard penetration tests conducted on geologically aged uncemented British sands. They have compared

the normalised SPT blow count $(N_1)_{60}$ for dry, geologically aged sands with that for young normally consolidated sands which are represented by values given by Terzaghi and Peck, and found that aged sands showed significantly higher values. They attributed this to the alteration of the micro fabric, caused by diagenetic processes. However when the tests were carried out below ground water table or in the capillary zone, N values obtained were much smaller. They did not offer any explanation for this phenomenon.

Baldi et al. (1988) discuss the effect of ageing on penetration resistance. They argue that penetration of any device causes considerable straining of surrounding sand and obliterates to a large extent the effects of ageing, and stress and strain history of the deposit. They report the results of many calibration chamber tests which show that strain hardening induced by overconsolidation and cyclic pre-stressing has little influence on the penetration resistance. They suggest that similar behaviour may be expected in the case of natural aged sands also. But they do not provide any experimental evidence in support of this.

Berardi et al. (1991) have commented on the effect of ageing on the penetration resistance of sands. They present the results of standard penetration resistance conducted on Holocene and Pleistocene deposits of Messina Strait Crossing in Italy. They report that SPT N values with in the Pleistocene strata are on an average only slightly higher than those of the Holocene materials. Nevertheless the stiffness of Pleistocene materials were considerably greater. They suggest that in the case of natural aged cohesionless deposits. due to secondary compression, early diagenesis, cementation and other environmental factors, the operational stiffness could be appreciably higher than that of the young normally consolidated deposits, while all parameters inferred from penetration tests show slight to moderate increase.

Troncoso (1994) has report the results of SPT and CPT tests conducted on deposits of silty sands in some tailings dams of Chile showing gain in penetration resistance with increasing age. They have commented that in these deposits formed by hydraulic fill methods, after sedimentation and consolidation the gain in strength has to be caused by seismic history and

ageing effects.

Thus while many researchers have reported ageing related increase in penetration resistance, Baldi et al. and Berardi et al. have advanced the position that ageing may not result in significant increase in strength. While they have not presented detailed experimental evidence, their arguments merit some consideration. Probably these contradictory views regarding the effect of ageing on penetration resistance may result from the different conceptions of ageing. Ageing may involve many phenomena and the effect of each on different properties may be different. Perhaps some insight into these can be obtained by a thorough understanding of the mechanisms involved.

Liquefaction Resistance

The work of Omote and Miyamura (1951) who presented data that showed a trend of less damage occurring in older sandy deposits during the Tokaido earthquake of 1944, is cited by Mulilis et al. (1977). Cassagrande (1975) suggest that for a proper assessment of the liquefaction susceptibility of natural sand deposits, whenever possible the age of the deposit should be determined, because there are indications that young alluvial sands are much more susceptible to actual liquefaction or development of large strains than old sediments that have been already subjected to too many severe earthquakes.

Mulilis et al. (1977) investigated the effect of ageing on the liquefaction resistance of sands by conducting undrained cyclic triaxial tests on isotropically consolidated specimens. A series of tests were carried out on reconstituted specimens of Monterey No. 0 sand, which were subjected to the initial effective confining stress for periods of 20 minutes, 1 day, 10 days and 100 days. The results were presented in terms of the cyclic stress ratio, required to cause 100% pore pressure response in 10 cycles of loading. They found that specimens tested after consolidating them for 20 minutes and 1 day, had almost identical values of the cyclic stress ratio. But with increasing age, this ratio was found to increase. The sample that was aged for 100 days showed an increase of about 26%. Mulilis et al. (1977) also conducted some tests on undisturbed samples from four different sand deposits which had been sub-

jected to the current overburden stress for periods ranging from about 55 to 2500 years. They also performed experiments on freshly deposited samples of these sands reconstituted at the same density. The undisturbed samples were found to have larger cyclic stress ratios in comparison to the reconstituted specimens. Mulilis et al. (1977) suggest that in view of sampling disturbances the actual values for undisturbed samples are likely to be still higher. Based on these results they conclude that a relatively short geologic time period of 2500 years, could be expected to increase the cyclic stress ratio by a factor of about 1.5 to 2.0.

Youd and Perkins (1978) consider age to be an important factor which influences the susceptibility of natural deposits to liquefaction during earthquakes. They present a qualitative estimate of the likelihood that various types of deposits may liquefy in terms of their age. Older deposits are less likely to liquefy than young deposits. These estimates were based on interpretation of published reports of earthquakes from all over the world. The work of Banerjee et al. (1979) who found that under sustained pressure the resistance to cyclic undrained loading of a gravel increased, is quoted by Tatsuoka et al. (1988).

Seed (1979) discusses some aspects of ageing in relation to susceptibility of ground to liquefaction. He considers the duration for which a deposit has been subjected to the overburden stress, as an important factor affecting the liquefaction and cyclic mobility characteristics. He also draws attention to the fact that cyclic resistance characteristics of medium dense and dense sands, determined from laboratory tests on undisturbed samples are often much lower than the in-situ values. He suggests that the loss of the effects of cementation developed during the geologic history, could be one reason for such differences in resistance.

Ishihara (1985) reviews some of the work done in Japan on cyclic strength of sands and presents data to show that undisturbed samples of natural deposits exhibit higher strengths compared to specimens of the same sand reconstituted to the same relative density. He suggests that this increase may be attributed to the effects of cementation or ageing which occurred during

the geologic history of these sands.

Tatsuoka et al. (1988) investigated the effect of sustained pressure on the liquefaction strength of sands. They performed cyclic undrained triaxial tests on loose and dense specimens of two kinds of sands which were subjected to sustained pressure for periods ranging from 0.1 hour to 68 days. They observed that specimens subjected to longer periods of sustained pressure showed an increase in strength. This increase in strength with time was larger for the sand containing some fines. It was found that the effect of 68 days of ageing was approximately equivalent to that of overconsolidation with an OCR of 2.

Troncoso (1994) reports that inspections of seismic damages in tailings dams in Chile indicate that in general old abandoned deposits behaved better than young deposits. He describes an ongoing long term research project, which attempts to monitor the evolution of pore pressure parameters with time in natural deposits, using field instrumentation. He states that although the data obtained was not sufficient to draw definite conclusions, there were indications to show that there was a tendency for lower pore pressure increase during more recent seismic events.

Locked Sands

Dusseault and Morgenstern (1979) introduced the term locked sands to differentiate a class of cohesionless materials from dense sands and sand stones. They described the origin and the distinguishing features of these materials. When quartz sands are subjected to overburden stress for long periods of time, if the dominant diagenetic processes are solution and quartz overgrowth formation, a dense uncemented fabric with a preponderance of interpenetrative contacts may result. The material still remains cohesionless, but the special fabric resulting from the diagenetic processes lead to a unique behaviour. The major characteristics of locked sands are—a porosity less than the minimum attainable in the laboratory, a quartzose composition, a diagenetic fabric and associated grain surface textures, generally older than quartenary, and little or no inter-granular cement.

Dusseault and Morgenstern (1979) conducted shear box tests to study the strength behaviour and the results showed that the behaviour was brittle with very small strains to failure. Cohesion was practically zero and the material had very high peak angle of shearing resistance and exceptionally large rates of dilation at failure. The large dilatancy was attributed to the inter-penetrative fabric. A disruption of these fabric returns the shear strength behaviour of the material to that of an ordinary sand. The failure envelope of locked sands are invariably curved because an increase in normal stress suppresses dilation in favour of shear across asperities.

Barton et al. (1986) carried out investigations on two British Tertiary sands to determine whether they were locked sands. These were uncemented quartzose sands with in-situ relative densities greater than 100%, but did not show the relatively large percentage of long and inter-penetrative contacts present in the North American locked sands as reported by Dusseault and Morgenstern (1979). The angles of shearing resistance were not as high as those of North American locked sands and the failure envelope did not show a significant curvature. Based on these results they concluded that there were some similarities and also differences in behaviour with North American locked sands and hence the British sands could be described as partially or weakly locked sands.

Palmer and Barton (1987) have investigated the nature and extent of diagenetic changes and the consequent effects on strength, in matrix-free, uncemented, clean British quartzose sands ranging in age from the Jurassic to the Recent. They conducted direct shear tests at normal stresses ranging from 50–900 kPa. The samples exhibited approximately linear envelopes over this stress range and showed increasing values of angle of shearing resistance and cohesion intercept with the age of the deposit. Though they found some evidence to suggest that at very low normal stresses the envelope may be curved, this aspect was not investigated in detail. The increase in strength of the deposit with age was ascribed to the progressive changes in types of contacts and increase in grain contact areas mainly brought about by pressure solution.

2.2.2 Ageing and Deformation Behaviour

Shear Modulus

Afifi and Woods (1971) investigated the effect of long term pressure on the shear modulus of soils including sands. They conducted resonant column tests on dry samples of three different sands which were subjected to confining stress for periods ranging from about 1 minute to 430 days. They observed that throughout this period shear modulus continued to increase. The variation of shear modulus with logarithm of time could be represented by two straight lines intersecting at about 1000 minutes. It was found that for the sands tested the increase in shear modulus per log cycle of time beyond 1000 minutes was about 2–5%. They also noted that, since the changes in void ratio during this period were extremely small, the contribution of reduction in void ratio towards this increase in modulus as would be predicted by the equations of Hardin and Richart (1963) would be practically nil. Afifi and Richart (1973) reported that time dependent increase in shear modulus is relatively unimportant for soils having $D_{50} > 0.04$mm. For these soils the increase in shear modulus per log cycle of time expressed as a percentage of the value at the end of 1000 minutes was found to be less than about 3%.

Jamiolkowski et al. (1988) present a comparison of, shear wave velocities calculated using an empirical relationship proposed by Ohta and Goto (1978) with those measured using the cross hole method, for four natural cohesionless deposits of Italy. The Ohta and Goto relationship expresses shear wave velocity as a function of standard penetration resistance, the depth of the deposit and two factors depending on age of the deposit and soil grading. It was found that the reliability of the empirical correlation decreases with increasing age of the deposit. One of the possible reasons considered for this discrepancy was that perhaps the effects of age and stress history were not properly accounted for.

Berardi et al. (1991) report some data in support of the view that ageing leads to an increase in shear modulus. The data pertains to the silica sand and gravel deposits at the Sicilian shore of the Messina Strait in Italy which

developed from laboratory tests on reconstituted samples may not be directly applicable to natural aged sand deposits.

Jamiolkowski and Manassero (1995) present data to quantify the influence of ageing on G_{max}. The rate of increase of G_{max} per log cycle of time is expressed as a percentage of the value at a time corresponding to the end of primary consolidation and is denoted by N_G. For silica sands N_G was found to vary from about 1.2 to 3.5. For a sand consisting of 50% quartz and 50% glauconite it was 3.9. For two carbonate sands they report values of 5.3 and 12.

Deformation Modulus

Daramola (1980) investigated the effect of consolidation age on the stress-strain behaviour of a sand by conducting drained triaxial compression tests. Four identical samples of Ham river sand were isotropically consolidated under the same stress. One sample was sheared immediately after consolidation and the others after 10, 30 and 152 days. It was found that while the behaviour of the samples sheared immediately and after 10 days were almost similar, specimens subjected to longer periods of sustained pressure showed significant changes in terms of stiffness and volume change behaviour. In the aged samples the tendency for volume decrease during shear was found to be smaller. Though the results showed some scatter, the trend of the data indicate that there is approximately a 50% increase in modulus per log cycle of time. Daramola suggested that this would imply that for a deposit whose age is 300 years, the modulus would be 3.5 times that of a freshly deposited sand.

Baldi et al. (1989) discuss the effect of age on the ratio of drained secant deformation modulus and cone penetration resistance of sands. They suggest that for normally consolidated aged deposits, the value of this ratio would be between that for overconsolidated and recent normally consolidated sands. This recommendation is based on the assumption that the response of aged sands would be similar to that of a slightly overconsolidated sand ($1 < \text{OCR} < 1.5$). The results of two screw plate load tests conducted on a natural sand

deposit which was at least 3000 years old were found to be in agreement with this.

Critical Stress and Constrained Modulus

Whitman et al. (1964) carried out some constant rate of loading confined compression tests in which the loading was interrupted for brief periods of time and then resumed. These tests showed that when the load was maintained constant, additional strains developed with time and up on reloading there was a relatively stiff response up to a certain critical stress.

Mesri et al. (1990) have demonstrated the development of a critical stress as a result of secondary compression. They conducted oedometer tests on saturated samples of a sand wherein the vertical stress was increased in increments allowing only primary consolidation. When the required stress was reached, the sample was allowed to undergo secondary compression for 1 day and then loading was resumed as before. The stress-strain curve clearly showed a critical stress up to which the behaviour was stiff, and beyond this stress the response was similar to an unaged specimen.

Coefficient of Earth Pressure at Rest

The in-situ state of stress has a profound influence on the strength and stress-deformation characteristics of soils and hence a correct assessment of in-situ horizontal effective stress is vital to the solution of many geotechnical engineering problems. Also some researchers have proposed that horizontal stress increases during ageing and this may partly account for some of the time dependent increase in strength and stiffness due to ageing. Therefore it is of considerable interest to examine the effect of ageing on the coefficient of earth pressure at rest (K_0). Unfortunately not much is known about the time dependence of K_0 for sands. The information available in the literature mostly pertains to clays. Nevertheless, considering the similarities between sands and clays regarding, the relationship between K_0 and the angle of shearing resistance, and also the effect of overconsolidation on K_0, it may be expected that the nature of the time dependence of K_0 would be at least

qualitatively similar. Hence the available information on the effect of time on K_0 for all soils is reviewed.

Schmertmann (1983) raised the question whether during secondary compression K_0 increases, decreases or remains constant. He also presented the results of a survey in which several professionals considered to be experts in the area of soil consolidation and related behaviour were polled on this subject. Of the 32 engineers who responded to the poll, 16 expected K_0 to increase with time, 4 expected K_0 to decrease, 9 considered K_0 to be independent of time and 4 stated that they did not know the answer. Schmertmann's paper clearly brought to light the existing gap in knowledge in this area and initiated further discussions on this subject.

The arguments and evidence in support of the time dependence or independence of K_0 may be classified into three broad categories. The first set consists of experimental investigations in which researchers have attempted to measure the variation of K_0 with time in long term laboratory tests. Another group has tried to resolve the problem theoretically, by developing mathematical relationships expressing K_0 as a function of time. The third category comprises heuristic reasoning based on the nature of soil behaviour.

Many researchers have conducted laboratory tests in which measurements of K_0 were made during secondary compression. Harris and Finno (1994) have summarized the results of such tests performed by several researchers. It is seen that tests carried out in triaxial cells show an increase in K_0 with time. However in the case of oedometer tests reported by four researchers, an increase in K_0 was noted only in one test. Some criticisms have been raised against the experimental procedures adopted in these tests. Holtz and Jamiolkowski (1985) have discussed the possibility of errors resulting from the lack of temperature control and failure to maintain a condition of zero lateral strain, and suggested that such errors may be partly responsible for some of the changes in K_0 observed in these tests. Harris and Finno (1994) noted that most tests showing an increase in K_0 with time were performed in the triaxial cell. In the K_0 triaxial test the condition of zero lateral strain is maintained by constantly adjusting the cell pressure so that the volumetric strain is equal

to the axial strain. Harris and Finno argued that, since several investigators have observed leakage to occur even in the most carefully conducted tests, failure to account for this could partly be a reason for the observed increase in K_0 with time. They pointed out that if at all K_0 changes with time, this will be small and the cumulative effect of all the errors may give a completely erroneous picture of the effect of time on K_0.

Holtz and Jamiolkowski (1985) present the results of the measurements of K_0 of a quartz sand in a large calibration chamber. The specimen was subjected to a constant vertical effective stress and the horizontal stress was constantly adjusted to maintain the diameter of the specimen constant. Measurements made over a period of 15 days indicated that variations in K_0 were insignificant and of a random nature.

Soydemir (1984) and Kavazanjian and Mitchell (1984) have presented theoretical relationships which show K_0 to increase with time. However as pointed out by Schmertmann (1984) and Harris and Finno (1994) such an approach entails the selection of a constitutive model in which the nature of time dependence is already specified. Therefore it is doubtful whether these relationships truly reflect the real behaviour.

Some arguments based on behavioural reasoning have been put forward to show that K_0 may increase with time. Kavazanjian and Mitchell (1984) suggested that the isotropic consolidation state with $K_0 = 1.0$ should represent a minimum energy state for the soil because of the absence of any global deviator stress. Particle movements during secondary compression should lead to a reduction of the global deviator stress. As vertical effective stress remains constant, a reduction of deviator stress can only result from a change in lateral stress. Hence they hypothesized that for normally consolidated and lightly overconsolidated soils with $K_0 < 1.0$, there would be an increase in K_0 with time. Similarly for heavily overconsolidated soils with $K_0 > 1.0$, a decrease in K_0 with time is likely. In both cases K_0 would ultimately tend to a value of 1.0. Holtz and Jamiolkowski (1985) in their discussion on this paper stated that their experience with sedimentary clay deposits of varying geologic ages did not indicate any tendency of K_0 approaching 1.0. In his

discussion Leonards (1985) also presented several arguments to show that there is no evidence to support the idea of K_0 approaching 1.0.

The foregoing discussion of the available information on the effect of time on K_0 show that a definite picture has not yet emerged. While there is some experimental evidence showing K_0 increasing with time, for these to be accepted without reservations it needs to be proved that experimental errors have not contributed to this increase. At the same time there are some experimental results showing K_0 to be independent of time and even some showing K_0 decreasing with time. Theoretical approaches have their own limitations and arguments based on behavioural reasoning may not be acceptable as conclusive. The issue can be resolved only by conducting careful experiments which are free from errors.

2.2.3 Ageing After Disturbance of Ground

There is a growing body of evidence which shows that following disturbance to ground there is often a time dependent increase in strength and stiffness. Usually these phenomena are also termed as ageing. In this subsection the information available on the time dependent increase in penetration resistance subsequent to ground modification and the increase in load capacity of piles with time after driving is reviewed.

Post Densification Penetration Resistance

Mitchell and Solymar (1984) describe the time dependent gain in strength of a freshly densified clean alluvial sand, in connection with the foundation densification for a dam in Nigeria. The upper layers were densified by vibro-compaction and the lower ones by blasting. Cone penetration tests were carried out before and at different times after densification. It was observed that immediately after blasting there was a reduction in penetration resistance, even though densification was indicated by ground settlements. With time, however, penetration resistance was found to increase over a period of several months and after a few weeks exceeded the original values. Similar

time dependent increase in penetration resistance was observed after vibro-compaction also, though the effects were somewhat less in comparison to that for blasting.

Dowding and Hryciw (1986) conducted a laboratory study of blast densification of saturated sand. Cone penetration tests performed at different times after blasting revealed that the resistance increased with time. They also carried out some control tests with out prior blasting, in which the time dependent increase in penetration resistance was monitored. They found that the increase in strength with time following blasting was considerably larger than that observed in the control experiments.

Schmertmann (1987) in his discussion on the paper by Mitchell and Solymar (1984) present data which shows the increase in cone resistance with time following deep dynamic compaction. Charlie et al. (1992) describe the results of cone penetration tests conducted 1, 3 and 18 weeks after blasting in a dense saturated alluvial sand deposit. Tests carried out 1 week after the blast showed a decrease in tip resistance and local friction. Tests performed after 3 weeks and 18 weeks revealed that with time there was an increase in tip resistance but local friction continued to decrease.

Increase in Pile Capacity After Driving

The phenomenon called set-up or soil freeze is well-known. This refers to the time dependent increase in load capacity of piles driven into soft or medium clays or loose saturated silts and silty sands and is generally considered to be caused by dissipation of pore pressures and re-consolidation within the zone disturbed by installation of the pile. Thus this effect is essentially due to an increase in effective stress with time in soils with poor drainage characteristics. But there is some evidence that set-up often occurs in highly permeable sands also, where it is believed to be caused by other processes.

Tavenas and Audy (1972) describe the significant time effects on the bearing capacity of piles, observed during a detailed testing programme of piles driven into a fairly homogeneous deposit of fine sand. Load tests were conducted at different intervals, for a period of about 50 days after driving.

Despite considerable scatter in the results, there was a consistent trend showing increase in ultimate bearing capacity for the first 15 to 20 days. At the end of this period pile capacity was about 50 to 80% higher than the value at the end of 12 hours. The authors state that, since a detailed investigation has shown the site to be fairly homogeneous and pore pressures induced by driving were expected to dissipate rapidly, the increase in pile capacity with time has to be related to the changes occurring in the sand structure around the pile.

York et al. (1994) report data showing time dependent increase in the load capacity of displacement piles driven into a medium dense deposit of medium to fine glacial sand. The set-up was calculated as the difference between the load capacities at the end of initial driving and either at the beginning of re-strike which was computed from a dynamic analysis or as determined by load tests. The data show that the pile capacity increased by 40 to 80% within 15 to 25 days, and it remained more or less constant after this period. They also suggest that the time required for dissipation of excess pore pressures would only be of the order of a few minutes and hence practically all the set-up takes place as the soil ages at constant effective stress.

It may be noted that such increase in pile capacity with time need not occur under all conditions. The opposite phenomenon of relaxation wherein the capacity decreases with time has also been reported for piles driven into dense sand (Yang 1970, York et al. 1994).

2.2.4 Mechanisms of Ageing

A rational conception of the nature and consequences of ageing can only follow from a thorough understanding of the actual mechanisms which cause this phenomenon. Elucidation of the mechanisms is also a necessary prerequisite for the development of mechanistic models of ageing. A detailed review of the arguments and evidence presented by several researchers for and against different plausible mechanisms of ageing is presented in this subsection.

Pore Pressures and Relative Density

Many researchers have raised the question whether the improvements in soil characteristics due to ageing result from dissipation of excess pore pressures or increase in relative density. There seems to be enough evidence in the literature to the contrary and there is a consensus among researchers that these phenomena cannot account for the ageing effect.

At a site where significant time dependent increase in bearing capacity of piles was recorded (Tavenas and Audy 1972), it was reported that, the sand was so pervious that complete dissipation of pore pressures could be expected to occur in a few hours time. Similarly York et al. (1994) suggest that at the site described by them, the pore pressures resulting from pile driving could be expected to dissipate in a very short time. But in both cases the load capacity of the piles continued to increase for a period of 15 to 20 days. Mitchell and Solymar (1984) report that, at a site where a considerable increase in penetration resistance occurred over a period of several months after blasting and vibro-compaction, calculations with even the most conservative assumptions indicated that all excess pore pressures generated would have been dissipated within a few minutes. The water level in an observation well returned to its pre-blast level by the end of first hour following a blast. Similar arguments are advanced by Charlie et al. (1992) in connection with blast densification of a sand deposit. All these researchers are unanimous in their opinion that dissipation of pore pressures could not account for ageing phenomena.

Ageing effects observed in the laboratory by several investigators (Mulilis et al. 1977, Daramola 1980, Tatsuoka et al. (1988), cannot be attributed to pore pressure effects, considering the durations involved. It may also be noted that ageing effects have been observed in the case of dry sands (Afifi and Woods 1971), which point towards mechanisms other than pore pressures. Dowding and Hryciw (1986) in their laboratory study of blast densification of sands observed penetration resistance increasing even after 15 days. They also suggest that pore pressures could not be responsible, as even the most conservative estimate indicated complete dissipation of pore

pressures within a minute. Thus an impressive amount of evidence both in the laboratory and the field, demonstrate that the dissipation of pore pressures is not a mechanism of ageing.

With regard to changes in relative density, most investigators are of the opinion that the increase in relative density during ageing period is so small that it cannot be expected to contribute in a significant way to the observed increase in stiffness and strength. Afifi and Woods (1971) report that the contribution of the time dependent increase in density to the total time dependent increase in shear modulus of sands was almost nil. They state that some mechanism or combinations of mechanisms other than the direct change in void ratio was responsible for the increase in shear modulus with time. Mulilis et al. (1977) report that the change in relative density due to a period of 100 days under sustained pressure was only about 0.4% and suggest that it is highly unlikely that this small increase in relative density was the cause for the observed 26% increase in undrained cyclic strength. Though Daramola (1980) did not state the changes in relative density during the ageing period. a comparison of the void ratios of the samples before shearing assuming that initial void ratios were almost same, indicates that these changes may not be significant. The difference in the end of consolidation void ratios of the sample which was sheared immediately and the one which was sheared after being subjected to the confining stress for a period of 152 days was only about 0.01. Tatsuoka et al. (1988) estimated that the increase in relative density for specimens during long term consolidation for a period of 68 days was only about 0.2%. But during this period there was a significant increase in the liquefaction resistance.

Experience with ageing phenomena after ground densification also point towards similar conclusions. Mitchell and Solymar (1984) state that there is no real increase in density during the ageing period. At the site described by them densification occurred immediately after blasting and the subsequent surface settlements were negligible. Time effects have been reported to occur after blasting even in cases were there was no net densification of the ground (Charlie et al. 1992).

There appears to be sufficient evidence to support the view that changes in relative density during long term consolidation in the laboratory and after densification of the ground are negligible. But in the case of locked sands there is a large increase in relative density. Typically, relative densities higher than 100% have been reported for these materials (Dusseault and Morgenstern 1978, Barton et al. 1986, Palmer and Barton 1987, Barton and Palmer 1989). Barton and Palmer found a good correlation between the age of the deposit and the relative density for geologically aged British sands. However it should be noted that there is a difference between ordinary sands and locked sands, with respect to the mechanisms of porosity reduction. During long-term consolidation in the laboratory and following disturbance of ground, secondary compression is the cause for the small reduction in porosity. But in the case of locked sands, the large reduction in porosity is believed to be brought about mainly by pressure solution at contacts between particles (Dusseault and Morgenstern 1978, Palmer and Barton 1987).

Explosion Generated Gases

It has been observed that immediately following blasting there is a reduction in penetration resistance and with time penetration resistance increases and may often exceed the pre-blast values (Mitchell and Solymar 1984, Dowding and Hryciw 1986, Charlie et al. 1992). Dowding and Hryciw (1986) put forward the hypothesis that the formation and dissipation of explosion generated gases may be responsible for these phenomena. Detonation of explosives produce large volumes of gas. Presence of these gas bubbles increases the compressibility of sand and this may be responsible for the reduction in penetration resistance immediately after blasting. With time the gases dissolve, migrate to the surface or are carried away by ground water and accordingly penetration resistance increases with time. This hypothesis may partly explain the time dependent phenomena in cases where explosive gases are a factor. But as pointed out by Mitchell and Solymar (1984) ageing effects have been observed in situations where gases are not a factor at all, and hence other mechanisms should be operating in those cases.

Cementation

Some investigators have proposed that the observed increase in strength and stiffness with time may be due to the formation of cohesive bonds resulting from deposition of cementing materials at inter-particle contacts. Mulilis et al. (1977) suggest that some form of cementation occurring at the contact points of sand grains, could be one possible reason for the increase in liquefaction resistance observed in specimens of sand which were subjected to sustained pressure. Mitchell and Solymar (1984) state that the most probable cause for the time dependent phenomena associated with ground densification could be the formation of silica acid gel films on particle surfaces and the precipitation of silica or other materials from solution or suspension as a cement at particle contacts. They have quoted the results of the research on the dissolution and precipitation rates for silica to show that the rates for amorphous silica are greater than those for crystalline quartz, and equilibrium if reached would occur in a few weeks or months time. They pointed out that, this was consistent with the durations involved in field observations of strength gain. They also report that chemical analysis of samples from the site where densification was carried out, indicated that sufficient material suitable for participation in cementation reactions were available. They concluded that while the available information was not sufficient to conclusively establish dissolution and precipitation reactions as the cause of the observed behaviour, all the evidence support them as the dominant mechanism.

Mitchell and Solymar (1984) have drawn attention to the work of Denisov and Reltov (1961) in support of the cementation hypothesis. Denisov and Reltov have described some of the investigations carried out by Russian researchers on chemical reactions on the surface of silicates. It was reported that, as a result of hydrolysis the superficial silicate layer disintegrates and this is accompanied by the formation of certain soluble compounds and silicic acid gel. Silicic acid gel has negligible solubility and remains at the surface, in the form of a very thin film which firmly adheres to the undisturbed silicate layer. These films have cementing properties and can form strong bonds between particles. Denisov and Reltov suggested that the gradual increase in

the shear strength of sands in hydraulically placed embankments may be due to these phenomena. Bonds between sand grains may be assumed to arise due to the gradual joining of the silicic acid gel films formed at the surface of the grains as a result of chemical interaction between water and silica.

Denisov and Reltov have also presented the results of experiments conducted to demonstrate the presence of gel films on sand grains and their effect on shear strength of sand. They measured the force required to dislodge a quartz particle resting on a quartz plate when both are kept immersed in water. It was shown that the adhesion between the particle and the plate increased with the duration of contact between the particle and the plate. Most of the increase occurred during the first 6 to 15 days. They explained this by assuming that the process of formation of gel films was completed during this period. Further solution of silica at the surface and the formation of gel associated with the process, was prevented by the silicic acid gel film formed.

Charlie et al. (1992) observed that the rate of increase of the normalised cone tip resistance with time following deposition or densification of sand, depends on the ambient temperature. They have analyzed the records of the time dependent increase in cone tip resistance for three cases of blasting, one each of vibro-compaction and dynamic compaction and one of placement of sand-fill. They found a constant rate of increase with the logarithm of time. They have related the value of this constant to the mean annual air temperature at the place. They observed a good correlation, with the logarithm of this constant being approximately linearly related to the temperature. Charlie et al. (1992) assume that the mean annual pore water temperature may be reasonably represented by the mean annual air temperature. Hence they suggest that the observed dependence of the rate of increase of normalised cone tip resistance on the mean annual air temperature, may support the idea of the involvement of chemical reactions as a cause of the formation of cementation bonds between particles.

Charlie et al. (1992) made another observation which may support the hypothesis of cementation. They found in their study of blast densification

of sand that, the decrease in cone tip resistance following blasting was 62% whereas local friction decreased only by 30%. Subsequently tip resistance was found to increase with time, while local friction and friction ratio continued to decrease. The work of Rad and Tumay (1986) show that, the greater the degree of cementation, the higher the tip resistance and lower the friction ratio. Thus the effect of cementation on tip resistance is more pronounced in comparison to that on local friction. Hence if cementation is the dominant mechanism of ageing it may be expected that, there should be a larger immediate reduction in tip resistance due to disturbance and a larger increase with time subsequently, in comparison to local friction. The results of Charlie et al. (1992) appears to partially support this view. It may be noted that while cementation should result in time dependent increase in local friction also, albeit at a lower rate, in this case it was found to decrease with time.

York et al. (1994) observed that the increase in pile capacity with time after completion of driving for some piles were lower than the general trend. They suggest that the lower density of the sand deposit in these cases could be the reason. They state that this explanation would be consistent with the studies of the increase in penetration resistance of sand fills with time by Dudler et al. (1968) who concluded that the effect of time was greater for dense sands and well-graded sands than for loose and uniform sands. It is well-known that dense and well-graded sands have a greater concentration of inter-particle contacts. According to York et al. (1994) this suggests that the sensitivity of an aged deposit and the corresponding recovery in strength following disturbance are proportional to the concentration of inter-particle contacts. A larger number of contacts would imply more opportunities for inter-particle bonds to form and hence these observations may support the idea of cementation as an important mechanism of ageing. At the same time it could be argued that the same could support the hypothesis that micro interlocking may be an important mechanism.

Some investigators have presented arguments against the possibility of cementation being the predominant mechanism of ageing. Dowding and Hryciw

(1986) in their laboratory study of blast densification of sand, observed that in many instances, at locations in the vicinity of detonation, there was a reduction in penetration resistance immediately after the blast. They noted that as the blasting was carried out within minutes after the deposition of sand, inter-particle cementation bonds could not have developed and hence the reduction in strength could not be attributed to the effects of cementation. They also noted that the increase in penetration resistance with time recorded in control tests without blasting were considerably less than the effects following blasting. According to Dowding and Hryciw (1986), this indicate that other mechanisms (in this case dissipation of explosion generated gases) must also contribute to the time dependent increase in penetration resistance after blasting.

Tatsuoka et al. (1988) have attempted to identify the mechanisms responsible for the increase in liquefaction strength of sands subjected to sustained pressure. They have compared the behaviour of samples subjected to long term consolidation and overconsolidation. They related the liquefaction strength of samples with the plastic axial strain induced as a result of overconsolidation or secondary compression. They observed a good correlation irrespective of whether the strain resulted from secondary compression or overconsolidation. It was argued that this similarity between aged and overconsolidated samples suggested that cementation could not be the cause for the increase in strength, because the short time periods involved in the consolidation stress history of overconsolidated samples (only 1 hour) were not sufficient for the formation of any inter-particle cohesive bonds due to cementation at particle contacts.

Mesri et al. (1990) advanced the argument that, since Daramola's tests (Daramola 1980) showed that sustained effective stress under drained conditions increases modulus up to rather large strains, a mechanism less brittle than cementation must be responsible for the increase in resistance to deformation. They noted that a hypothesis that implies cementing bonds forming at grain contacts of clean silica sands in a matter of weeks or months would preclude the existence of any uncemented natural sand deposits. According

to them this did not appear to be reasonable.

Schmertmann (1991) suggests that the nature of locked sands was contrary to the ideas that cementation was the dominant mechanism of ageing. During geologic ageing, such sands developed little or no cohesive bonding. Schmertmann's work also tries to answer the question whether ageing related improvements in strength and stiffness result from an increase in the frictional or cohesive components of strength. If cementation indeed occurs, then it must lead to an increase in cohesion. Schmertmann used a special type of triaxial test developed by him and denoted as the IDS test. to separate the basic cohesive and frictional components of shear strength. Results of such tests on aged samples of clays showed that there was no increase in the cohesion component due to ageing whereas the frictional component increased. However since similar results for sands are not available. it is not known whether the same is true in this case also.

Secondary Compression

Many researchers have proposed that changes in fabric induced by secondary compression, may be an important mechanism responsible for the increase in strength and stiffness with time. Mulilis et al. (1977) suggest that secondary compression which might cause a slight movement of the sand grains into a more stable position could be a possible reason for the increase in liquefaction strength due to long term consolidation. Tatsuoka et al. (1988) related the increase in cyclic strength of samples which were subjected to long term consolidation and overconsolidation to the plastic axial strains resulting from these processes and found the relationship to be similar. According to them this suggested that the increase in strength was mainly due to a slight movement of the sand grains into a more stable position.

Mesri et al. (1990) examine several mechanisms of ageing which have been suggested by various researchers. They state that, though they could not provide any direct evidence against the cementation hypothesis, a more rational explanation of drained ageing of clean sands might be in terms of larger frictional resistance and horizontal stress. They hypothesize that during sec-

ondary compression, the continued rearrangement of sand particles leads to enhanced macro-interlocking of sand grains and micro-interlocking of grain surface roughness and this results in an increase in stiffness and effective horizontal stress. Any structural disturbance caused by vibro-compaction, blasting, or dynamic compaction, even though may not result in an increase in vertical effective stress, could cause pronounced secondary compression effects than that achieved by an ideally static increase in effective vertical stress. They present the results of oedometer tests in which more pronounced secondary compression effects were observed in samples which were first densified and then allowed to age.

Schmertmann (1991) also discusses a number of mechanisms proposed to explain ageing phenomena. He presents the results of IDS tests which show an increase in the frictional component of strength with time, whereas the cohesive component remains more or less constant. After examining all the available evidence, he concludes that the strengthening effects result from an increase in frictional resistance. Schmertmann also suggests that the small slippages of particles during secondary compression might result in enhanced interlocking and considers this to be the major cause for the increase in strength and stiffness. In support of this, he cites the work of Daramola (1980) who showed that volume decrease during shear decreased as the age of the sample increased. This was attributed by Schmertmann to the increased interlocking resulting from secondary compression.

Mesri (1993) describes ageing tests on mineral to mineral contacts, wherein quartz-quartz and mica-mica contacts were submerged in distilled water and subjected to normal stress for different periods of time and then sheared in a direct shear apparatus. Though ageing did not appear to influence the angle of sliding friction ϕ_μ, a significant increase in initial stiffness was observed. Micrographs of the quartz surface showed the surface to be rough at a microscopic level. According to Mesri, this supports the hypothesis that an important mechanism of ageing is the increase in frictional resistance resulting from the enhanced interlocking of particle surface roughness. However the mere existence of surface roughness may not be sufficient to prove the

predominance of interlocking as a mechanism of ageing. It may be argued that the results obtained by Mesri could also be ascribed to the effects of cementation. But as cementation is likely to increase the shear strength of the contact, if time effects resulted in an increase in the initial stiffness and not in the shear strength. then it is an indication that cementation was unlikely to be the cause.

Increase in Horizontal Effective Stress

Mesri et al. (1990) and Schmertmann (1991) have pointed out the possibility of the effective horizontal stress increasing during secondary compression and have suggested that this may be partly responsible for the increase in strength and stiffness with age. But changes in horizontal stress may be particularly significant in ageing phenomena after disturbance of the ground.

Schmertmann (1987) suggests the possibility that ground improvement methods such as vibro-flotation and blasting, may temporarily create conditions of very low lateral stress in the region surrounding a vibrating probe or the point of an explosion, after the withdrawal of the probe or dissipation of the gas pressures after the explosion. This might explain the immediate reduction in penetration resistance. The time dependent recovery of horizontal stress might at least partly account for the subsequent increase in q_c. Hryciw and Dowding (1988) report the results of dilatometer tests performed after blast densification of a sand deposit. Their results indicated a decrease in the effective horizontal stress immediately after the blast. However they do not present any data on changes in horizontal stress with time.

In contrast to the idea that after ground densification lateral stress may increase with time, Charlie et al. (1992) report the results of measurements which indicate that in some situations the reverse may happen. Cone penetration tests conducted 1 week and 68 weeks after blasting of a sand deposit revealed that local friction decreased by 39% during this period. But there was no significant change in soil density during this period. Charlie et al. (1992) suggest that this decrease in local friction with time indicated that lateral stress decreased with time. This contradicts Schmertmann's contention

that lateral stress may increase with time.

There is some evidence that piles installed in dense sands may experience a reduction in load capacity with time, an effect termed as relaxation (Yang 1970, York et al. 1994). York et al. (1994) attributed relaxation to the dissipation of negative pore pressures induced by driving, and the relief of large lateral stresses induced by the tendency of the sand mass around the pile to dilate. The fact that such time dependent changes in horizontal stress are possible, suggests that under favourable conditions increase in horizontal stress may occur and contribute to increase in strength and stiffness.

Pressure Solution and Crystal Over-growth

Diagenetic processes leading to large reduction in porosity and a change in the predominant type of inter-particle contact from tangential to long,inter-penetrative and sutured, and resulting in very high relative densities, have been proposed as the mechanism by which ordinary sands are transformed into locked sands. Dusseault and Morgenstern (1979) describe the distinguishing characteristics of locked sands as—a highly quartzose mineralogy and rugose surface texture; a highly interlocked structure; very high relative densities usually exceeding 100%; lack of interstitial cement; and a stress-strain behaviour characterized by pronounced brittleness, absence of cohesion, very high peak angle of shearing resistance, exceptionally high dilation rates at failure, residual angles of shearing resistance not much different from those of ordinary sands and pronounced curvature of the failure envelope.

The high relative densities of locked sands imply that large reductions in porosities have taken place, after the deposition of these sands. Dusseault and Morgenstern (1979) suggest that even small amounts of solutioning which could occur at grain contacts where the stresses are the highest, might result in significant reductions in porosity. Some reduction in porosity could also occur due to the recrystallisation of silica from solution.

Palmer and Barton (1987) describe the main diagenetic processes that control porosity reduction in sediments, in connection with their research on British locked sands. According to them the main processes are—mechanical

compaction due to overburden pressure, mechanical compaction due to seismic disturbances over a geologic time scale, cementation and chemical compaction. After a thorough examination of various relevant factors like the nature of sands, the mineral strength and texture, absence of interstitial cement, stress levels involved and the very large reductions in porosity, they conclude that the first three processes could not have been the cause. They state that inter-granular pressure solution at grain contacts resulting in material mass transfer, was the main cause of the large reduction in porosity during the long history of these deposits.

But the very high relative densities alone are not sufficient to explain the extremely large peak angle of shearing resistance observed in some locked sands. For Athabasca oil sands Dusseault and Morgenstern (1978) report values as high as 55–65°. Dusseault and Morgenstern (1978, 1979) explain such high strengths in terms of the exceptionally high rates of dilation at failure for these materials. A coarse Ottawa sand in a dense state had dilation rates at failure which were only a fraction of those shown by the oil sands. Such high rates of dilation were attributed to the highly interlocked fabric of these sands, which resulted from a preponderance of long and inter-penetrative contacts whereas ordinary sands typically have tangential contacts. This transformation from predominantly tangential type of contacts to other types was considered to be brought about by the processes of pressure solution and recrystallisation acting throughout the long geologic history of these deposits. These sands have a three dimensional interlocked structure which promote the propagation of dilation beyond the failure plane. thereby resulting in pronounced dilatancy at relatively low levels of confining stresses.

Dusseault and Morgenstern (1978, 1979) also explain some of the other characteristics of locked sands. The pronounced brittleness of these materials is because of the large reduction in the dilatational component beyond the peak. The marked curvature of the failure envelope is due to the suppression of dilatancy at higher values of normal stress. As the high rates of dilation result from interlocking brought about by inter-penetrative and sutured contacts which can be sheared with relative ease, the tendency of these sands

to dilate is readily suppressed by increasing the normal stress. The residual angles of shearing resistance of locked sands (30–35°), though not drastically different from other quartz sands, are still higher than those of rounded sands like Ottawa sand. This was attributed to the grain surface rugosity and grain angularity accentuated by solution pits and quartz over-growths.

2.2.5 Prediction of Ageing Improvement

Though a large amount of data on time dependent increase in strength and stiffness of soils both in the laboratory and the field exists, attempts at modelling these phenomena with a view to develop rational predictive methodologies have been limited. The main impediment to progress in this area seems to be the lack of a clear understanding of the mechanisms and processes involved. Mitchell and Solymar (1984) state that the processes involved in ageing could be considered as early diagenesis from a geologic point of view and in principle their rates and magnitude of improvement could be estimated using theoretical models. However, they suggest that till the governing kinetic rate laws and parameters are known or defined, the magnitude of sand sensitivity and rate and amount of time dependent strength gain after disturbance or densification will have to be determined by direct measurement. It may be noted that Mitchell and Solymar consider cementation to be the principal mechanism of ageing.

The currently available approaches to prediction of the rate and magnitude of the increase in strength and stiffness with time may be classified into two main types. Firstly there are purely empirical relationships based on the observed rates of increase in the laboratory. The second group of methods consists of relationships based on the assumption that secondary compression is the dominant mechanism of ageing. These have been proposed mostly by Mesri and his co-workers.

Empirical Methods

Empirical relationships have been proposed to express the increase in the maximum shear modulus due to long term consolidation. These equations are based on experimental observations which show that beyond a certain time the shear modulus versus log time relationship can be represented by a straight line (Afifi and Woods 1971). The general form of these equations may be expressed as follows (Kokusho et al. 1982, Mesri and Choi 1983).

$$G_{max(t)} = G_{max(t_R)}[1 + N_G \log(t/t_R)] \tag{2.4}$$

where $G_{max(t)}$ is the initial shear modulus at time $t > t_R$. $G_{max(t_R)}$ is the initial shear modulus at time $t = t_R$, and N_G is a dimensionless parameter indicating the rate of increase of G_{max} per log cycle of time which is defined as $\mathrm{N}_G = \Delta G_{max}/G_{max(t_R)}$, where ΔG_{max} is the increment in G_{max} per log cycle of time.

By knowing the value of N_G for a soil the increase in G_{max} at any time can be estimated from the above equation. Jamiolkowski and Manassero (1995) present values of N_G for different types of soils including some sands.

Methods Based on C_α/C_c Concept

Mesri and his co-workers have over the years made significant contributions to the understanding of the phenomenon of secondary compression and its effect on the subsequent response of soils. They have proposed a number of equations which facilitates quantitative evaluation of the effects of secondary compression on many characteristics like critical stress resulting from secondary compression, increase in undrained shear strength, shear modulus and coefficient of earth pressure at rest. Mesri (1987) has summarized many of the significant findings from the research on secondary compression and its effect on behaviour of clays.

A fundamental assumption invoked in the derivation of many of these equations is the notion that soil behaviour is a function of the effective confining stress and that in soils that have undergone some amount of secondary compression the value of stress which govern behaviour is either an equivalent

stress which is the value on the virgin compression line corresponding to the same void ratio or a critical stress which is the apparent pre-consolidation stress resulting from secondary compression. The other key concept is the relationship between the compressibility with respect to time expressed in terms of the secondary compression index (C_α) and the compressibility with respect to effective stress expressed in terms of the compression index (C_c). Mesri and his co-workers have shown that for a soil the ratio C_α/C_c is a constant, provided the appropriate values of these parameters are used (Mesri and Castro 1987). The value of this constant for each major types of soils lies in a very small range.

Mesri et al. (1990) hypothesize that secondary compression is the predominant mechanism resulting in increase of strength and stiffness of sands during ageing, and extend the concepts and relationships developed for cohesive soils to sands. They present equations for increase in shear modulus, constrained modulus, coefficient of earth pressure at rest and cone tip resistance.

Modifying the equation originally proposed by Mesri and Choi (1983) for cohesive soils assuming a linear relationship between shear modulus and the critical stress resulting from secondary compression, Mesri et al. (1990) propose the following equation for sands, for which shear modulus is assumed to be directly proportional to the square root of the critical stress.

$$N_G = \exp\left(1.15\frac{\frac{C_\alpha}{C_c}}{1-\frac{C_r}{C_c}}\right) - 1 \tag{2.5}$$

where $C_c = -\Delta e/\Delta \log \sigma'_v$ is the compression index beyond the critical stress, and $C_r = -\Delta e/\Delta \log \sigma'_v$ is the compression index up to the critical stress.

The following equation was proposed for estimating the increase in constrained modulus during secondary compression of clean sands, under a constant vertical effective stress of σ'_{v0}.

$$\frac{M}{M_p} = \frac{1}{C_r/C_c}\left(\frac{t}{t_p}\right)^{\frac{C_D C_\alpha}{C_c}} \tag{2.6}$$

where M_p is the constrained modulus at σ'_{v0} at time t_p corresponding to the end of primary consolidation, M is the constrained modulus at σ'_{v0} for time

$t > t_p$, and C_D is a parameter reflecting any densification by such mechanisms as vibration and blasting, which are not related to the ideally static increase in effective vertical stress.

For the prediction of the coefficient of lateral earth pressure at rest K_0 during secondary compression Mesri et al. (1990) propose the following relationship.

$$K_0 = K_{0p}\left(\frac{t}{t_p}\right)^{\frac{C_D C_\alpha}{C_c}} \tag{2.7}$$

where K_{0p} is the coefficient of earth pressure at rest at time t_p corresponding to the end of primary consolidation, and K_0 is the coefficient of earth pressure at rest during secondary compression at time $t > t_p$.

They also propose a tentative relationship for estimation of the increase in cone penetration resistance with time by assuming a direct relationship between q_c and $\sqrt{M\sigma'_h}$. With this assumption, equations for M and K_0 lead to the following expression.

$$\frac{q_c}{(q_c)_R} = \left(\frac{t}{t_R}\right)^{\frac{C_D C_\alpha}{C_c}} \tag{2.8}$$

where $(q_c)_R$ is the reference cone penetration resistance at a reference time $t_R > t_p$, and q_c is the cone resistance at any time $t > t_R$.

Mesri et al. (1990) suggest a procedure for determining the empirical constant C_D, from the results of oedometer tests in which M and M_p were measured. The effect of densification in the field due to blasting, vibro-compaction, dynamic compaction etc was simulated in the laboratory by densification of the sample by tapping the sides of the container and then allowing secondary compression to occur. The values of C_D determined from such tests were deemed to be appropriate to predict the increase in q_c occurring after ground densification.

Mesri et al. (1990) made an attempt to fit Equation 2.8 to the data from Jebba dam site described by Mitchell and Solymar(1984). They report that while in some cases a good fit was observed, in other cases the results were less satisfactory. They attributed this to scatter in q_c data related to variation in

ground conditions, variations in ground modification procedures, systematic problems in post densification q_c measurements and different values of t_R.

A major difference between the relationships proposed by Mesri et al. (1990) and the earlier work of Mesri and his co-workers on clays is the introduction of the empirical parameter C_D. They state that the parameter C_D reflects any densification caused by such disturbances due to vibration and blasting, which are not related to the ideally static increase in effective stress and is a measure of the potential for stiffness increase that is developed by the ground modification effort. They suggest that C_D may be influenced by the method of ground modification, stress levels and also whether a macro-arching effect was introduced. Mesri (1993) state that the parameter C_D is a measure of the potential for increase in inter-particle resistance with time. Thus it appears that the parameter C_D need not be associated with disturbance effects alone, since increase in resistance with time has also been observed during secondary compression following static increase in effective stress.

Chapter 3

Materials and Methods

In this chapter a brief description of the materials used in the present investigation is given. Available information on the mineralogical and morphological characteristics of some sands are presented. Various physical characteristics of the materials used. as determined in the present study are also presented. Most of the experiments conducted in this study consists of common laboratory tests using standard equipment and procedures. Therefore only the salient features of the experimental methods used are described.

3.1 Materials

Six different materials were used in this study, of which five are of geologic origin. Experiments were performed on three natural sands (Ganga, Fine Kalpi and Japanese) and two artificial sands (Standard sand and Crushed rock). These materials represent a wide spectrum of morphological characteristics and this was one of the important motivations for the selection of these sands. Some experiments were also carried out on Steel bearing balls which could be considered in many respects as a limiting case for granular materials.

3.1.1 Description of Materials Used

Herein a brief description of the materials used is presented. The discussion on Ganga, Kalpi and Standard sands is adapted from Rahim (1989).

Ganga sand is a natural sand collected from the flood plains of river Ganga near Kanpur. Ganga sand is predominantly angular with minor amounts of flaky as well as rounded grains. The mineral composition is mostly quartz and feldspar with some mica and small amounts of calcite (carbonate) and chlorite. Fragments of gneiss and quartzite also occur in minor quantities. The quartz grains are quite transparent and often noticed to be fractured and sometimes coated with iron oxide. The feldspars are not as transparent as quartz and are partly altered. Presence of mica reflects the contribution of phyllitic material from the foot hills of Himalayas as one of the possible sources. The carbonate constituents may be due to associated carbonate formation in the Himalayan sequence.

Fine Kalpi sand consists of the fraction of Kalpi sand passing the 0.425 mm sieve. Kalpi sand is a natural river sand from the Yamuna river system at Kalpi. The natural Kalpi sand is well-graded, relatively coarse and very angular. Quartz and feldspar present in equal proportions. together constitute nearly 80% of the total mineral content. The carbonate content in the form of calcite could be as high as 18% and occurs in the form of a matrix agglomerated with quartz and feldspar grains. When treated with mild acid this matrix dissolves, leaving the silicate particles that constituted the nucleus for the growth of carbonate. In addition carbonate is also present in a minor quantity in the form of limestone grains. The sand used in the present study was obtained by sieving the natural Kalpi sand through a 0.425 mm sieve to remove the coarser particles and has been termed Fine Kalpi sand to distinguish it from the natural Kalpi sand. Despite the differences in particle size distribution, it is believed that with respect to morphological and mineralogical aspects, Fine Kalpi sand is similar to Kalpi sand. Therefore it may be assumed that the above description of the morphological and mineralogical characteristics of Kalpi sand may be generally applicable to Fine Kalpi sand also.

Standard sand is an artificial sand obtained from Ottawa (USA) and is uniformly graded and consists entirely of quartz. The grains are coarse, nearly spherical and wellrounded. The particles are opaque and are endowed with a frosted surface, a characteristic which may have been acquired during the manufacturing process.

Japanese sand is a fine beach sand from Japan. The silt content of this sand is somewhat higher than that of other sands used in the present investigation. Two tube samples of this sand were obtained from Japan and the material from these were used in this study. The material contained some coarse fragments of shells and decayed wood and these were removed by sieving.

Crushed rock was obtained by separating the relatively finer particles from a lot of crushed rock used as coarse aggregate in the production of concrete. The fraction passing through 3.35 mm sieve and retained on 2.80 mm sieve was used. This particular size was chosen so that the particle size is comparable to that of Steel bearing balls. A visual examination revealed the particles to be bulky and very angular.

Steel bearing balls are perfectly spherical in shape with a diameter of 3.175 mm. The surfaces of these appear to be smooth.

3.1.2 Properties of Materials

The particle size distribution of Ganga, Fine Kalpi, Standard and Japanese sands is shown in Figure 3.1. The mineralogical composition of Ganga, Kalpi and Standard sands has been determined by Rahim (1989) and this is presented in Table 3.1. The morphological characteristics of Ganga. Kalpi and Standard sands given in Table 3.2 are also taken from the same reference. The various physical properties of the materials used in the present study are presented in Table 3.3. As the required equipment was not available. the minimum void ratio of the materials was not determined in this study. The values of e_{min} for Ganga and Standard sands shown are from Shahu (1988).

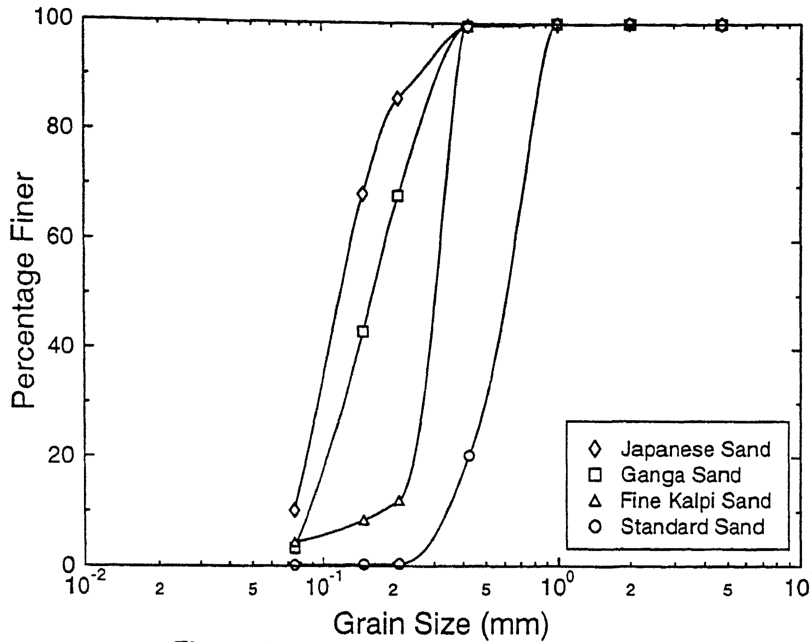

Figure 3.1: Particle size distribution of sands

Table 3.1: Mineralogical composition of some sands, after Rahim (1989)

Sand	Percentage Mineral Content				
	Quartz	Feldspar	Mica	Carbonate	Chlorite
Ganga	60–65	20–25	8–10	2–3	2–3
Kalpi	40	40	1–2	18	0
Standard	100	0	0	0	0

Table 3.2: Morphological characteristics of some sands, after Rahim (1989)

Sand	Average Shape Factor	Average Sphericity	Average Roundness
Ganga	0.539	0.678	0.15–0.25
Kalpi	0.596	0.698	0.10–0.20
Standard	0.766	0.81	0.78–0.80

Table 3.3: Physical properties of materials used

Material	D_{60} mm	D_{50} mm	D_{10} mm	C_u	Fines %	G_s	e_{max}	e_{min}
Ganga	0.19	0.165	0.09	2.11	3.3	2.68	1.14	0.57
Fine Kalpi	0.27	0.26	0.18	1.50	4.4	2.66	1.04	—
Standard	0.50	0.48	0.36	1.39	0.2	2.65	0.74	0.45
Japanese	0.13	0.12	0.074	1.75	10.2	2.67	1.34	—
Crushed Rock	$2.80 < D < 3.35$				0	2.63	1.31	—
Steel Bearing Balls	$D = 3.175$				0	7.81	0.70	—

3.2 Experimental Methods

Experiments conducted in this study consists of oedometer, direct shear and fall cone tests and tests to determine various properties of the materials used.

3.2.1 Determination of Material Characteristics

Particle Size Distribution and Specific Gravity

The particle size distribution was determined by conducting sieve analysis in accordance with standard test procedures. Specific gravity (G_s) was determined by using specific gravity bottle.

Maximum Void Ratio

The maximum void ratio e_{max} was determined by pouring the material into a container of 100 mm diameter and 60 mm height, through a funnel as per the following procedure. The tip of the funnel was kept touching the bottom of the container at its centre and the funnel was filled with sand. Then the funnel was gradually raised vertically, allowing sand to flow through the opening, with a height of fall that was kept to a minimum. While pouring sand, the funnel was not moved horizontally. Thus sand was deposited in the container in the form of a conical heap, with the particles slowly moving down the slope from the apex to the bottom. The clearance between the tip

of the funnel and the apex of the conical heap of sand in the container was throughout maintained at a value which was just sufficient to allow sand to flow through the funnel opening. When the container was full, the surface was levelled carefully with a straight edge and the weight of the sand was determined accurately. The volume of the container was determined by filling it with mercury. Knowing the weight, volume and specific gravity of the sand, void ratio was calculated and this was taken as e_{max}.

It was observed that, if while pouring sand the funnel was moved to and fro laterally, a denser packing was obtained. The loosest packing was achieved when the funnel was raised gradually and vertically with out any lateral movement. It is believed that when the particles are allowed to slowly roll or slide down the slopes of the conical heap, they get the maximum opportunity for mutual interference and thus interlocking develops to the fullest extent possible. Also since the height of fall is practically zero and movements occur mostly along the surface of the conical heap, the momentum of the particles is quite small and there is little compaction. Thus this method may give the loosest possible packing.

3.2.2 Preparation of Samples

Most of the tests were conducted on dry specimens of sands, prepared by air pluviation. In this method sand was poured through a funnel, keeping the height of fall constant. While pouring, the container was moved to and fro laterally so that the surface of sand in the container was practically level. The required density was obtained by varying the height of fall and/or the diameter of the funnel opening. Some fall cone tests were conducted on specimens which were compacted to different densities by vibrating them on a shaker for the required amount of time.

3.2.3 Oedometer Tests

Oedometer tests were performed using conventional apparatus. The oedometer cell consisted of a rigid stainless steel ring with polished inner surface,

a bottom plate and a loading cap with a ball seating. Loads were applied by suspending weights on a lever arm loading system. All tests were of the incremental loading type.

Cornforth (1974) discusses some of the common errors associated with conventional oedometer testing. These include — apparatus compliance, side friction, locally high void spaces at the contact with the cell, difficulty in achieving plane contact with upper and lower stones and problems in the accurate determination of relative densities because of the small dimensions of the specimen. The present study consists mostly of comparative tests in which the relative effects of certain characteristics or processes on some aspects of behaviour were investigated. Therefore the consequences of some of these errors may not be as serious as in the case when absolute magnitudes of soil properties are of interest. However wherever it was practical, care was taken to reduce these errors as far as possible.

The apparatus was calibrated for its own deformations by using a dummy specimen made of brass and having approximately the same dimensions as the actual samples. The loads were applied in the same sequence as in the actual tests and resulting vertical deformations were measured. For each loading sequence, 3 to 5 repetitions were made and the average curve obtained was used for calibration. The actual compression of the sand specimen was obtained by subtracting the apparatus deformation obtained from this average curve from the total measured vertical deformation. To reduce side friction, a coating of silicone grease was applied to the inner surface of the container. However the effect of applying the grease was not investigated. The thickness of the specimens were measured using a dial gauge graduated to 0.025 mm and the weight was determined to an accuracy of 0.1 gm.

Initially tests were carried out on specimens of 60 mm diameter and 20 mm thickness. It was observed that in the range of stresses applied ($<$ 200 kPa), there was considerable scatter in the results. One possible reason could be that in this range of stresses, initial differences in densities and fabrics of specimens have considerable influence. Another source of scatter may be the errors involved in the measurement of small deformations.

The dial gauge used was graduated to 0.002 mm and further improvements in resolution is difficult. Therefore the other alternative of increasing the magnitude of deformation by increasing the sample thickness was tried. Also it was thought that by carrying out the tests at higher stresses scatter could be reduced. Hence most of the investigations were carried out at a vertical stress of 1600 kPa or more on specimens having a diameter of 75.7 mm and thickness of 46.0 to 46.5 mm. Although the operating stress levels were higher than those normally encountered in common field situations, it is felt that the results of these tests may be extended to lower stress levels with a reasonable degree of confidence.

In all tests, loads were applied in increments of equal duration. However the usual procedure of using a load increment ratio of unity was not followed, nor was any attempt made to keep a constant value. Whenever it was required to have a well-defined stress strain curve, small load increments were used. Since the objective was to examine the mechanisms involved and to compare different materials, it is believed that such a procedure is justified. In most of the tests the duration of each load increment was 5 minutes, though some tests were also conducted with different durations. All the tests were performed on dry specimens, because of the ease of specimen preparation and also for better control on densities of the specimens.

Another source of uncertainty could be the effect of temperature variations. It is well-known that in the case of clays, temperature has some influence on many aspects of behaviour including secondary compression. In the present study due to lack of required facilities no attempt was made to control the room temperature. However it was thought that the errors resulting from temperature fluctuations may not be as serious as in the case of clays. Since the tests were conducted on dry sands there is no viscous flow involved. Moreover the main factors controlling particle movements are friction and particle interference. It is doubtful whether these processes are influenced to any significant degree by the normal fluctuations of room temperature.

3.2.4 Direct Shear Tests

The direct shear tests were carried out in a 60 mm square shear box using a machine of conventional design. The normal loads were applied through a lever arm loading system on which weights were placed. The specimens were sheared at a constant rate of 0.25 mm per minute by a motorized drive unit. The horizontal displacements of the box were measured using a dial gauge graduated to 0.01 mm and the vertical displacements using one graduated to 0.002 mm. The shear force was measured using a proving ring. The thickness of the specimens were measured using a dial gauge graduated to 0.025 mm.

3.2.5 Fall Cone Tests

Fall cone tests were conducted using a commercially available apparatus used for the determination of liquid limit of cohesive soils. to which some modifications were made. A schematic arrangement of the setup is shown in Figure 3.2. It essentially consists of—a container holding the specimen; the cone assembly consisting of a right circular cone, a hollow aluminium rod to which the cone is attached by a threaded connection and weights that could be fixed to the top of the rod so that the total weight of the cone assembly could be varied; a frame to hold the cone assembly in place and to ensure the verticality of its downward motion; and a dial gauge for measuring the penetration of the cone into the sand. Most tests were carried out in a container having a diameter of 100 mm and a height of 60 mm. Some tests were also conducted in a smaller container of diameter 55 mm and height 39 mm.Tests were performed with cones made of stainless steel and perspex.

Tests were carried out according to the following procedure. Specimens were prepared at different void ratios, either by air pluviation or vibration and the top surface was carefully levelled using a straight edge. The container was positioned below the cone and the cone assembly was gradually brought down so that the tip of the cone just touched the surface of the specimen. The dial reading corresponding to this position was noted. The plunger of the dial gauge was lifted so that it was no longer in contact with the top of

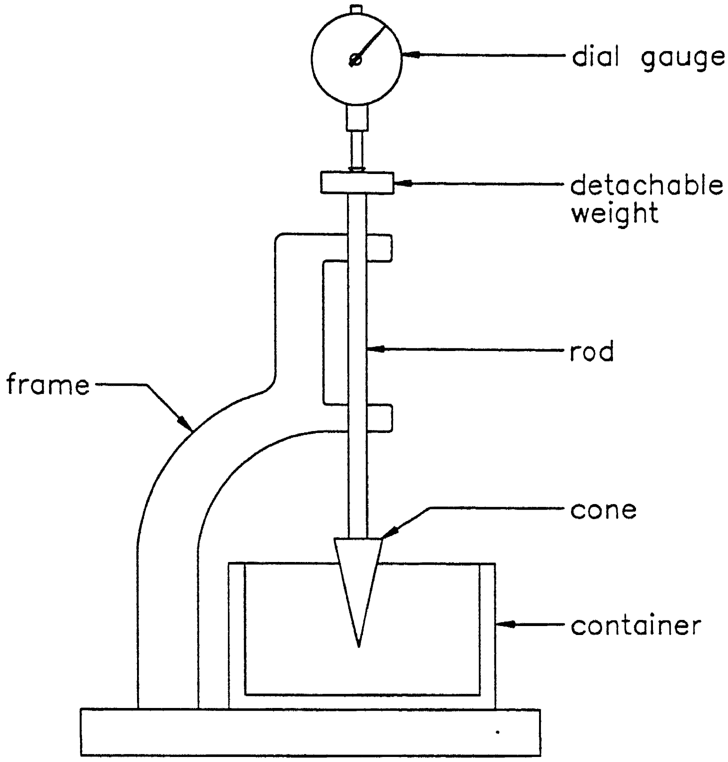

Figure 3.2: Experimental set-up for fall cone test

the cone assembly and was held at this position. Then the cone assembly was released and allowed to penetrate into the sand under its own weight. The plunger of the dial gauge was slowly brought down so that it rested on the top of the cone assembly and the reading was recorded. The penetration of the cone into the sand was obtained as the difference between the two readings. Void ratio of the specimen was calculated from its weight measured to an accuracy of 0.1g and the volume of the container measured by filling it with mercury.

Chapter 4

Results and Discussion

In this chapter, the significant results obtained in the course of the present study are presented and discussed. These include findings from the experimental investigations and also some inferences which are deduced from an analysis of the information available in literature. The content is organized in two sections, the first dealing with aspects related to morphology and the second with ageing.

4.1 Morphology and Engineering Behaviour

4.1.1 Morphology and e_{max}

There is ample evidence in literature to show that, for a constant value of the uniformity coefficient (C_u), both the maximum and minimum void ratios (e_{max} and e_{min}) decrease as roundness (R) increases. This was most convincingly demonstrated by Youd (1973), who presented a set of remarkably well-defined relationships between R and the limiting void ratios. Youd suggested that in view of the many deficiencies associated with the commonly used tests to determine the limiting void ratios, a better procedure would be to measure R and C_u and to obtain e_{max} and e_{min} from these relationships. But considering the difficulties involved in the measurement of R, it is doubtful whether geotechnical engineers would be willing to replace the

relatively simple and familiar tests currently in use for the determination of e_{max} and e_{min}, with a more cumbersome procedure.

Perhaps a more meaningful and practical question to ask is whether the above relationships can be utilized to make inferences regarding particle morphology, once e_{max} and C_u are known. This problem was considered by De Jaeger (1994), who proposed that as the limiting void ratios are primarily functions of grain shape and C_u and since the influence of C_u can easily be evaluated, e_{max} may be used as a shape index. But when e_{max} itself is used as a shape index, the effect of morphology on behaviour may often be masked by the influence of C_u. For example, consider the effect of particle shape on the constant volume friction angle (ϕ_{cv}). The available information shows that as R increases ϕ_{cv} decreases, whereas an increase in C_u results in a higher ϕ_{cv}. On the other hand, e_{max} decreases as both R and C_u increase. This suggests that a meaningful relationship between e_{max} and ϕ_{cv} may not exist. Hence a better approach may be to obtain R from e_{max} and then relate the required properties to R.

If the picture depicted by Youd (1973) is valid for all sands, then it provides a simple but powerful technique for estimating R from e_{max}. But the results of a more extensive investigation by De Jaeger (1991, 1994), indicate that the situation is far more complex. The data presented by Youd (1973) and De Jaeger (1991) for sand fractions with C_u of 1.4 and 1.2 respectively, are shown in Figure 4.1. It is evident from the figure that, while Youd's data display a well-defined relationship, De Jaeger's data show considerable scatter even though the general trend is somewhat similar in both cases. Therefore it is doubtful whether a reasonably accurate estimate of R can be obtained from e_{max}.

Dickin (1973), Youd (1973) and De Jaeger (1994) state that, for identical grain shapes the effect of the mean particle size on e_{max} is not significant. To examine this aspect, the data presented by De Jaeger (1994) was interpreted in terms of particle size, as shown in Figure 4.1. It is clear that an appreciable influence of particle size is discernible in the case of coarser particles ($D_{50} > 0.6$ mm). Hence consideration of the effect of particle size could lead

to a better estimate of R from e_{max}. A part of the scatter in the relationship between R and e_{max}, could be due to differences in the distribution of roundness although the mean value may be the same. For example, the average value of R for a sand consisting of equal amounts of very angular and rounded grains would be approximately the same as that of a sand having equal proportions of subrounded and subangular grains. But it is not clear whether both sands would have the same e_{max}. It may also be noted that particle shape is an extremely complex property and aspects of shape other than roundness, could also exert a significant influence on e_{max}.

Thus it appears that, though for a given C_u, e_{max} and R do show a definite trend, considerable scatter is involved and hence it may not be possible to obtain a reasonably accurate estimate of R from these relationships. However, by allowing for the effect of particle size a significant reduction in this scatter is facilitated. These relationships when used judiciously could provide a measure of the order of magnitude of particle angularity. In view of the profound influence of roundness on the behaviour of sands and the complexities involved in its measurement, even an approximate estimate of R could be of great practical value.

4.1.2 Morphology and Effect of Method of Specimen Preparation

A majority of the laboratory investigations on the behaviour of sands, including common strength and deformation tests, model tests on sand beds, calibration chamber tests and centrifuge modelling are carried out on reconstituted specimens. A variety of methods are used to place the specimens at the required relative density — varying the height of fall and rate of pouring in pluviation or by compacting the sand by tamping, impact or vibration. The review of some of the investigations on the effect of method of deposition on behaviour presented in Chapter 2, show that specimens of a sand prepared at the same void ratio but by different methods, could exhibit significant differences in behaviour presumably due to variations in the resulting fabrics.

imum. Another method is the tilting method devised by Kolbuszewski, wherein sand is placed in a graduated measuring jar and is then rapidly turned upside down several times. e_{min} is usually determined by vibration, though other methods of compaction involving impact or repeated shear straining have been suggested. Many of these methods for the determination of e_{max} and e_{min} have some of the following limitations.

- Procedures for the determination of e_{max} are not standardized and hence repeatability is often poor (Youd 1973).

- e_{max} is measured under practically zero vertical stress, while e_{min} is measured after vibration under a specified vertical stress and hence the definitions of these parameters are not consistent (Sheng and Randolph 1994).

- e_{max} is usually determined by pouring and e_{min} by vibration. As the two methods lead to different fabrics and behaviour, it is not consistent to use e_{max} and e_{min} obtained in such a manner as reference states for computation of relative density.

- Compaction of sands using vibration with a surcharge or methods involving impact, may result in some particle modification and consequently in alteration of the nature of the sand during the test.

It may be noted that the procedures involved in the determination e_{max} and e_{min} constitute the operational definition of these parameters, apart from supplying the numerical values. Thus an ideal method should lead to a physically meaningful and consistent operational definition and should consist of experimental procedures which are simple, repeatable and applicable to a wide variety of sands. In this context it may be noted that the main concern is not to find out the absolute maximum and minimum void ratios at which a sand could be deposited, but to determine parameters to serve as reference states for the computation of relative density and as an index of particle morphology. Hence the best method need not necessarily be the one giving the absolute maximum or minimum values, but the one which most closely

satisfies the above requirements and functions. In the present work efforts were made to develop better methods for the determination of e_{max} and e_{min}.

Fall Cone Test for e_{max}

In the case of fine-grained soils, procedures based on the fall cone test have resulted in a physically meaningful operational definition of liquid and plastic limits and experimental methods that are simple and standardized. In this study an attempt was made to develop a method based on fall cone test, for the determination of e_{max} of sands.

The apparatus and procedures for conducting fall cone tests were described in Chapter 3. A perspex cone with an apex angle of 30° and a brass container of 100 mm diameter and 60 mm height were used. The total weight of the cone assembly was 60.5 g. Samples of Standard, Fine Kalpi and Ganga sands were prepared at different void ratios by pluviation and the penetration was measured. The results obtained are presented in Figures 4.4, 4.5 and 4.6 for Standard, Fine Kalpi and Ganga sands respectively, in terms of a plot of the reciprocal of the measured penetration (d) against the void ratio (e). The relationship between $1/d$ and e is found to be linear and the equation of these straight lines for each sand, obtained by least squares regression are also shown in the corresponding figures.

An examination of these figures reveals that, for each sand the value of the void ratio corresponding to $d = 50$mm ($1/d = 0.02$) is approximately equal to the e_{max} for that sand. It may be noted that the values of e_{max} determined by pouring the sand through a funnel according to the procedure outlined in Chapter 3, are 0.74, 1.04 and 1.14 for Standard, Fine Kalpi and Ganga sands respectively. This suggests the possibility of a new operational definition of e_{max} as the void ratio at which all sands have a specified penetration measured in a fall cone test. For the conditions adopted in this study this value of penetration may be taken as 50 mm. Thus by conducting fall cone tests at several void ratios the linear $1/d$ versus e relationships can be established and from the equation of the resulting straight line, e_{max} can be obtained as the void ratio corresponding to $d = 50$ mm. The values of e_{max} calculated

according to this procedure are 0.75, 1.045 and 1.10 for Standard, Fine Kalpi and Ganga sands respectively, which are found to be in good agreement with the values determined by pouring. Thus the fall cone test which is used for the determination of liquid limit of fine-grained soils, is shown to be a satisfactory procedure for the determination of e_{max} of sands also. It may be noted that in the case of the standard fall cone test for fine-grained soils, the liquid limit is operationally defined as the water content at which the penetration is 20 mm.

The possibility of a one-point method for e_{max} as in the case of liquid limit was also explored. The fact that the slopes of the linear relationships between $1/d$ and e for the three sands tested are approximately same (0.0805,0.0792 and 0.0869 for Standard, Fine Kalpi and Ganga sands respectively), suggests that such a simplification is indeed feasible. An average value of the slope, assumed to be applicable to all sands was calculated as 0.0822. Using this value for the slope and the value of d corresponding to e_{max} as 50 mm, the following expression was derived.

$$e_{max} = e + \frac{12.16}{d} - 0.243 \tag{4.1}$$

where d is the measured penetration in mm.

Hence if a fall cone test is conducted at a void ratio e and the measured penetration is d, then e_{max} can be calculated from the above equation. The validity of the one-point method was examined for the three sands mentioned above and in addition for a Japanese sand also. For each sand five repetitions were made at a particular void ratio and the average value of the void ratio and penetration were substituted into Equation 4.1 to compute the value of e_{max}. The results are presented in Table 4.1, where the values of e_{max} determined by all the three methods (pouring, detailed fall cone and one-point fall cone) are presented for comparison. It is seen that the one-point method yields results which are practically identical to those from the detailed procedure and also are sufficiently close to those determined by pouring. Thus the method is shown to give satisfactory results for four different sands which practically cover the full range of e_{max} values that are normally encountered.

Table 4.1: Comparison of e_{max} determined by pouring and fall cone tests

Sand	e_{max}		
	Pouring	Fall cone test	
		Detailed	One-point
Standard	0.74	0.75	0.75
Fine Kalpi	1.04	1.045	1.037
Ganga	1.14	1.10	1.11
Japanese	1.34	—	1.31

The fall cone test for e_{max} would have certain advantages over the other procedures currently in use. Firstly, it leads to an operational definition of e_{max} that is physically more meaningful since the criterion is based on penetration resistance and hence on the shearing resistance of the sand. Secondly, because of the functional similarity between liquid limit of clays and e_{max} of sands, an operational definition of e_{max} which is similar to that of liquid limit is aesthetically appealing. Finally, the test procedures and equipment are simple and standardized and repeatability is likely to be better. But the method suffers from a serious limitation because it may not be applicable to sands containing very coarse particles.

It should be noted that, apriori there is no reason to believe that sands with widely ranging characteristics should have the same penetration at a void ratio corresponding to e_{max} determined using a conventional procedure (say by pouring through a funnel). The proposed method is based on the observation that in the neighbourhood of e_{max} penetration changes rapidly with e and though the actual value of d corresponding to the conventional e_{max} could be different for different sands, the difference between the value of e corresponding to d = 50 mm and the conventional e_{max} is quite small. This fact is taken advantage of to propose a new operational definition of e_{max}.

The fundamental assumption involved in the one-point method is that the slope of $1/d$ versus e relationship is same for all sands. This implies

that for a given change in void ratio, all sands exhibit the same change in penetration resistance. As void ratio decreases vertical stress increases, but because of the increase in angle of shearing resistance horizontal stress would decrease. Hence if it is assumed that in the fall cone test the mean effective principal stress (P') is same at all void ratios, a change in void ratio is equivalent to a change in the state parameter (ψ) proposed by Been and Jefferies (1985). Then the above assumption reduces to the statement that for the same change in state parameter all sands have the same change in penetration resistance. Been and Jefferies (1985) found that many properties of sands could be correlated to the state parameter. However it should be noted that they had observed some differences between the behaviour of angular sands and that of subangular and subrounded sands having the same value of the state parameter. But it is felt that as the operating stress levels in the fall cone tests are extremely small there is no particle modification and hence these deviations if any are likely to be small. Therefore the assumption made in the one-point fall cone method appears to be reasonable, at least for the purpose of determining e_{max}.

Determination of e_{min}

Attempts made in the present study to develop criteria for e_{min} based on fall cone tests were not successful. The nature of the void ratio versus penetration relationships precluded the formulation of a satisfactory operational definition based on such criteria. Therefore a different approach was adopted in this case. The available evidence show that both e_{max} and e_{min} are functions of particle shape, mean particle size and gradation. Since both e_{max} and e_{min} basically represent packing characteristics, it may be assumed that the functional dependence of both these parameters on the particle characteristics are similar in nature. If this is true, then a good correlation between e_{max} and e_{min} should exist.

In fact many researchers in the past have suggested that for most sands, the difference between the maximum and minimum porosities ($n_{max} - n_{min}$) is approximately a constant. Alyanak (1961), Kolbuszewski and Frederick

(1963), Dickin (1973) and De Jaeger (1994) report values of 9.5, 10.0–12.0, 12.5 and 9.4% respectively. But a simpler approach may be to relate e_{max} and e_{min} directly and this has been attempted using published data for a large number of sands.

The data for poorly graded, non-micaceous sands ($C_u < 4$) is shown in Figure 4.7 where e_{min} is plotted against e_{max}. For the various sand fractions studied by Alyanak (1961), Youd (1973) and De Jaeger (1991), an excellent correlation between e_{max} and e_{min} may be observed. While there are differences between the different sets of data presumably due to variations in the test procedures, for each set of data there is a consistent linear relationship between e_{max} and e_{min}. The data collected from literature for a number of other sands is also presented in Figure 4.7 and a similar trend is seen, even though the scatter is some what more pronounced. The slightly larger scatter in this case may be attributed to the fact that the data has been gathered from a number of sources and thus is a reflection of the variations in the test procedures used by different researchers.

On the other hand, for well-graded sands and micaceous sands a more complex relationship between e_{max} and e_{min} is indicated. These sands have a lower ratio of e_{min} to e_{max} in comparison to poorly graded and non-micaceous sands. This may be explained as follows. Large values of C_u may cause a greater reduction in e_{min}, in comparison to the effect on e_{max}. Presence of mica will lead to higher values of e_{max} due to bridging action but when compacted under stress may result in a lower e_{min}.

The available evidence suggest that for most poorly graded sands, the relationship between e_{max} and e_{min} may be expressed as

$$e_{min} = \alpha \, e_{max} \tag{4.2}$$

Equation 4.2 may be adopted as a simple operational definition of e_{min}. Thus once e_{max} is determined (say by the one-point fall cone method), e_{min} is easily computed using Equation 4.2. It is proposed that a value of $\alpha = 0.62$ may be used since this is the average value for most natural sands for which data is available in literature. With $\alpha = 0.62$, the value of $n_{max} - n_{min}$ computed

using Equation 4.2 lies between 11 and 12%, for e_{max} in the range of 0.7–1.4 and hence is with in the range reported in literature.

4.1.4 Morphology and ϕ_{cv}

The constant volume friction angle (ϕ_{cv}) of sands is the value of the angle of shearing resistance mobilized at the critical state in a drained test. It is also considered to be equal to the steady state angle of shearing resistance (ϕ'_{ss}) corresponding to the steady state in an undrained test (Negussey et al. 1988). ϕ_{cv} represents a lower bound of shearing resistance and is of great relevance to many problems including prediction of steady state shear strengths, interpretation of penetration tests and formulation of flow rules to describe the plastic deformation of sands. Hence a thorough understanding of the factors controlling ϕ_{cv} is of considerable interest.

The available information on the effect of particle morphology on ϕ_{cv} was reviewed in Chapter 2. The effect of roundness was most convincingly portrayed by Stroud (1989) who summarized some of the data then available in the form of a graph between ϕ_{cv} and R. The data for predominantly quartz sands from Stroud (1989) together with additional data from Castro (1969), Castro et al. (1982), Sladen (1985), Negussey et al. (1988) and Pandey (1993) are presented in Figure 4.8 where ϕ_{cv} is plotted against R. Though the data has considerable scatter, the average trend for poorly graded sands can reasonably be represented by the relationship shown in the figure. It may be seen that changes in roundness alone could result in a variation in ϕ_{cv} of the order of 6°. a major part of which occurs in the very angular to subangular range. Well-graded sands, gravels and boulders have a larger ϕ_{cv} in comparison to poorly graded sands.

The effect of particle shape and size on the angle of repose of rock-fills is clearly brought out by the graph presented by Stephenson (1979) based on data reported by Simons and Albertson (1960), which is reproduced in Figure 4.9. For the very large particles the effect of shape is almost completely over-shadowed by that of size. But for sand-sized particles angularity is more significant, with the change in the angle of repose resulting from differences

in angularity being about 5.5°. It may be noted that the angle of repose of dry sands is considered to be approximately equal to ϕ_{cv} (Cornforth 1973). Hence the data shown in Figure 4.9 may be presumed to be applicable to ϕ_{cv} also.

Rowe (1962) considers the shearing resistance of sands to comprise three components—inter-particle sliding friction, dilatancy and particle rearrangement at constant volume. Lee and Seed (1967) suggest that at high stresses the phenomenon of particle crushing may also be important. At the critical state there is no dilatancy and particle crushing is likely to be insignificant and hence ϕ_{cv} may be assumed to consist of the contributions from sliding friction and particle rearrangement at constant volume. If the existence of a true inter-particle friction angle (ϕ_{μ}), which is a function of the mineralogy and surface chemistry and independent of the size and shape characteristics of the particles is posited, then differences in ϕ_{cv} as evident in Figure 4.8 may be regarded as arising from variations in the component due to particle rearrangement. This would imply that the rearrangement component is maximum in the case of angular sands and minimum in the case of rounded sands. It is interesting to note that this trend is contradictory to the statement of Rowe (1962, 1971) that the contribution from particle rearrangement could be expected to be maximum in the case of rounded sands. Figures 4.8 and 4.9 suggest that particle size and C_u also influence the rearrangement component.

Further insight into the nature of particle rearrangement is facilitated by consideration of other factors that could have some influence. These include relative density and the boundary conditions of the test. Rowe (1962, 1971) found that when the mobilized friction angle measured in the triaxial test is corrected for the effect of dilatancy, the resulting friction angle (ϕ'_f) lies in the range $\phi_{\mu} \leq \phi'_f \leq \phi_{cv}$. In the case of initially dense samples at low strain levels, $\phi'_f \approx \phi_{\mu}$ and for initially dense samples at large strains and initially loose samples at all strains, $\phi'_f \approx \phi_{cv}$. Negussey et al. (1988) observed that the mobilized angle of shearing resistance corresponding to the point of maximum volume contraction (a transient constant volume condi-

tion at which the angle of shearing resistance may be assumed to consist of the contributions from sliding friction and rearrangement only) decreased as relative density increased. Thus the rearrangement component is seen to be a maximum in the case of loose sands. Rowe (1962, 1971) found that for the plane strain condition $\phi'_f \approx \phi_{cv}$, irrespective of strain level and packing. Hence rearrangement appears to be more in the case of plane strain than in triaxial tests. The explanation offered by Rowe for these observations is based on the notion of a critical angle of sliding which corresponds to the condition of minimum dissipation of energy. When the number of contacts at which sliding takes place is relatively large, in general the instantaneous angles at which particles slide is different from this critical angle and hence $\phi'_f > \phi_\mu$.

Perhaps another way of looking at the phenomenon of particle rearrangement is to consider the fact that in any test specimen there would be spatial variation of local void ratio. Hence at the critical state even though the overall volume remains constant, there could be dilation and contraction at various points throughout the specimen. Therefore samples with a more non-uniform distribution of void ratio are likely to have a higher rearrangement component. In this context it is of some interest to note that the factors which give rise to larger standard deviation of void ratio have been reported to be—high angularity (Bhatia and Soliman 1990), low relative density (Oda 1977 and Bhatia and Soliman 1990) and non-uniform particle size distribution (Oda 1977). Under these conditions the particle rearrangement component is also high (Figure 4.8, Rowe 1962, 1971 and Negussey et al. 1988). Hence the factors leading to larger standard deviation of void ratio and greater rearrangement component seem to be closely associated. Thus it appears that the higher ϕ_{cv} in the case of angular sands could be attributed to a larger contribution from particle rearrangement and this may be related to the more non-uniform distribution of void ratio in this case.

4.1.5 Prediction of Angle of Shearing Resistance

Prediction of the drained angle of shearing resistance (ϕ') is an important pre-requisite for the satisfactory solution of design problems involving sands, where considerations of stability are important. The traditional practice of relating ϕ' to the relative density has not been completely satisfactory because of the inability to incorporate the important influence of stress level. The realization of the inadequacies of relative density has prompted researchers to search for a better state variable combining the influence of relative density and confining stress. Approaches towards the prediction of ϕ', involving two such state variables—the relative dilatancy index (Bolton 1986) and the state parameter (Been and Jefferies 1985), were reviewed in Chapter 2.

Bolton (1986) defines a relative dilatancy index of the form $I_R = I_D(Q - \ln P'_f) - R^*$, where I_D is the relative density, P'_f is the mean normal effective stress at failure and Q and R^* are empirical constants. By examining published test data for a large number of sands, Bolton arrived at the conclusion that the dilatational component of shearing resistance could be correlated to I_R, by taking $Q = 10$ and $R^* = 1$. These correlations were found to be applicable in the range $0 < I_R < 4$. Thus according to Bolton, for triaxial compression ϕ' is given by

$$\phi' = \phi_{cv} + 3\ (I_D(10 - \ln P'_f) - 1) \tag{4.3}$$

Apparently Bolton did not consider various characteristics of sands including morphology to be important in the range of stresses in which the above equation was found to give reasonable results. But it is desirable that any method should take into account the important influence of particle morphology. Though in principle this is feasible in Bolton's method by selecting appropriate values of Q and R^*, it is not clear as to how these empirical constants could be easily determined in practice.

On the other hand, for the state parameter proposed by Been and Jefferies (1985) the reference state is the steady state which reflects various characteristics of sands including particle morphology. Therefore the state parameter

approach is likely to be more successful in accounting for the effects of particle shape on behaviour. Following this approach ϕ' may be expressed as the sum of the constant volume friction angle (ϕ_{cv}) and a dilatancy component which is an empirical function of the state parameter (ψ).

$$\phi' = \phi_{cv} + \mathcal{F}(\psi) \tag{4.4}$$

In the present work, a method for the prediction of the angle of shearing resistance which has some similarities to the state parameter approach is proposed. This is based on the observation that, for a sand at a particular value of confining stress the product of the void ratio and the tangent of the effective angle of shearing resistance ($e \tan \phi'$) is a constant. This is implied in the work of Cassagrande (cited by Zolkov and Wiseman 1965) who presented typical trends of the variation of ϕ' with void ratio for rounded and subangular sands. Again Vesic (1963) found that, for Chattahoochee River sand with in a sufficiently narrow range of confining stress $e \tan \phi'$ was a constant. To verify whether this observation is applicable to all sands. published data on ϕ' for various sands at different void ratios were examined. In Figure 4.10, $e \tan \phi'$ for a variety of sands is plotted against I_D and it is seen that $e \tan \phi'$ remains reasonably constant over a wide range of relative densities. Thus it appears that for a given sand at a particular confining stress $e \tan \phi'$ may be considered to be a constant.

The value of this constant would in general be a function of the confining stress and the nature of the sand, especially its morphology. Hence if this constant can be expressed as a function of these factors, a simple relationship that would allow prediction of ϕ' is easily derived. If it is assumed that for a sand with an initial void ratio which is equal to the value of the void ratio at steady state (e_{ss}) corresponding to the existing confining stress, ϕ' is equal to the steady state angle of shearing resistance (ϕ'_{ss}), then the value of the above constant can be expressed as $e_{ss} \tan \phi'_{ss}$. Steady state and critical state are considered to be practically identical for sands (Sladen et al. 1985, Negussey et al. 1988). Hence the above constant may also assumed to be equal to $e_{cr} \tan \phi_{cv}$, where e_{cr} is the critical void ratio corresponding to the

initial confining stress. Thus for a sand at any value of initial void ratio e and initial confining stress, $e\tan\phi' = e_{ss}\tan\phi'_{ss} = e_{cr}\tan\phi_{cv}$.

Except for extremely dense sands, there would usually be a decrease in volume during early stages of shearing. Hence even for a sand which is initially at a void ratio equal to e_{cr}, there would first be a volume decrease and then dilation to the critical state and therefore a small contribution from dilatancy is likely. So the assumption that for a sand with an initial void ratio equal to the critical state void ratio at the same confining stress $\phi' = \phi_{cv}$, is not strictly correct. Similar considerations apply to pore pressure changes during undrained shear. Nevertheless it is proposed that for $e \leq e_{cr}$, the angle of shearing resistance may be estimated using the following equation.

$$\phi' = \tan^{-1}(\frac{e_{ss}\tan\phi'_{ss}}{e}) = \tan^{-1}(\frac{e_{cr}\tan\phi_{cv}}{e}) \qquad (4.5)$$

For $e > e_{cr}$, it may be assumed that $\phi' = \phi_{cv}$.

The validity of Equation 4.5 was examined using data presented by Lee and Seed (1967) who conducted drained triaxial compression tests on samples of Sacramento River and Ottawa sands having $I_D = 1$, at different values of confining stress. For Sacramento River sand with $e = 0.61$ and $\phi_{cv} = 33.5°$, ϕ' was predicted using values of e_{cr} reported by Lee and Seed (1967). As these authors did not present e_{ss} or e_{cr} data for Ottawa sand, in this case e_{ss} was calculated using the equation of the steady state line ($e_{ss} = 0.754 - 0.028\log P'$, where P' is in kPa) proposed by Been et al. (1987). In Figure 4.11 the measured values of ϕ' at various values of initial confining stress are compared with the values predicted using Equation 4.5. For Ottawa sand the predictions are quite good with the maximum error being only about 1°. In the case of Sacramento River sand for stresses less than 1000 kPa, predicted values of ϕ' are quite close to the measured values, but at higher stresses Equation 4.5 considerably under-predicts ϕ'. This discrepancy could in part be due to the effect of particle crushing. Lee and Seed (1967) observed measurable crushing at a stress of 1000 kPa and at higher stress levels intensity of crushing was more pronounced. Thus Equation 4.5 appears to work reasonably well when there is no alteration of particle characteristics induced by crushing.

A comparison of the predicted values of the dilatational component of the angle of shearing resistance ($\phi' - \phi_{cv}$) by Equation 4.3 proposed by Bolton (1986) and Equation 4.5 is presented in Figure 4.12. The predictions were made for Mine Tailings and Banding #6 sand using data from Castro et al. (1982) and for Toyoura sand using data from Ishihara (1993), for stresses of 100 and 600 kPa. It should however be noted that in Bolton's equation these stresses correspond to the mean principal normal effective stress at failure whereas in the present method these refer to the initial confining stress. It is evident from the figure that except for Mine Tailings at 600 kPa, the values predicted by Equation 4.5 lie with in the range predicted by Bolton. In the case of Mine Tailings the proposed equation predicts a large reduction in ϕ' at higher stresses (which seems to be reasonable in view of the angular nature of the material), while Bolton's method does not indicate such a possibility.

The effect of stress level on ϕ' for these three sands (Mine Tailings, Banding #6 sand and Toyoura sand) at $I_D = 1$ as predicted by Equation 4.5 is illustrated in Figure 4.13. The subrounded Banding #6 sand shows the least sensitivity to stress level, with subangular Toyoura sand being slightly more susceptible. The angular Mine Tailings exhibits the maximum dependency of ϕ' on stress level. Thus the expected trend of the effect of particle morphology on the nature of the variation of ϕ' with confining stress is clearly brought out by Equation 4.5.

The value of $e \tan \phi'$ for a sand would be a function of the nature of the sand and the magnitude of the confining stress. To illustrate the effect of particle shape on $e \tan \phi'$ comparisons should be made at the same stress level. For this purpose $e \tan \phi'$ for various sands at extremely low levels of confining stress may assumed to be equal to $e_0 \tan \phi_{cv}$, where e_0 is the void ratio on the steady state line corresponding to $P' = 0$. An examination of published data reveals that there is no unique relationship between e_{max} and e_0. However, based on the limited data available it is proposed that e_0 for most sands may be assumed to be approximately equal to the void ratio corresponding to $I_D = 0.1$. The value of $e_0 \tan \phi_{cv}$ was computed for poorly graded and predominantly quartz sands using published data and is

plotted against roundness in Figure 4.14. It may be seen that roundness has a marked influence with $e_0 \tan \phi_{cv}$ decreasing as R increases.

4.1.6 Morphology and N_γ versus ϕ' Relationships

Results of plate load tests on Chattahoochee River and Ganga sands presented by Vesic (1963) and Shahu (1988) respectively, indicate that at low levels of confining stress angular sands have considerably larger values of N_γ than would be predicted by the available theoretical relationships. In the present study the effect of morphology on N_γ was investigated using the results of fall cone tests on angular and rounded sands. For this purpose a method for the analysis of fall cone test was developed as described below.

In the following a simple model for the fall cone test, based on bearing capacity considerations is derived. Consider a cone with an apex angle (β) and supporting a total load of W which includes the weight of the cone and other attachments. Let d_0 be the depth of embedment for which the ultimate bearing capacity of the cone would be equal to W. If the cone is embedded to a depth of d_0 in a layer of sand with unit weight γ, then by equating the ultimate bearing capacity to the bearing stress, the following relationships are obtained.

$$\frac{W}{\pi(d_0 \tan(\beta/2))^2} = 0.5\ \gamma\ 2d_0 \tan(\beta/2)\ N_\gamma \tag{4.6}$$

$$W = \pi N_\gamma \tan^3(\beta/2)\gamma d_0^3 = K\gamma d_0^3 \tag{4.7}$$

Here $K = \pi N_\gamma tan^3(\beta/2)$ may be considered as a cone factor similar to the one used in the analysis of fall cone tests in clays (Hansbo 1957). But it should be noted that unlike in the case of clays where for a given cone K is considered to be a constant, for sands K is not a constant. From the above equation N_γ may be expressed as

$$N_\gamma = \frac{W}{\pi\gamma d_0^3 \tan^3(\beta/2)} \tag{4.8}$$

The above equations were derived from considerations of statics. But the fall cone test is a dynamic test and when the cone is released from rest, it

would initially accelerate and then would finally come to rest at a depth d which would be greater than d_0. By extending the dynamic analysis of the fall cone test in clays carried out by Hansbo (1957) and Houlsby (1982) to the case of sands, a relationship between d and d_0 may be derived.

If it is assumed that at any depth of penetration z the resistance offered by the sand is $K\gamma z^3$ (see Equation 4.7), then the equation of motion of the cone may be written as

$$\frac{W}{g}\frac{d^2z}{dt^2} = W - K\gamma z^3 \tag{4.9}$$

where g is the acceleration due to gravity and z is the penetration below the surface of the sand at any instant of time t. This equation may be rewritten as

$$v\frac{dv}{dz} = g - \frac{Kg\gamma z^3}{W} \tag{4.10}$$

where $v = \frac{dz}{dt}$ is the velocity of motion at any instant of time. Integrating both sides of Equation 4.10 and applying the boundary condition $v = 0$ at $z = 0$, the following equation is obtained.

$$v^2 = 2gz - \frac{Kg\gamma z^4}{2W} \tag{4.11}$$

Putting $v = 0$, the depth d at which the cone comes to rest is obtained as

$$d = \sqrt[3]{\frac{4W}{K\gamma}} \tag{4.12}$$

Substituting $W = K\gamma d_0^3$ in Equation 4.12 yields

$$d = \sqrt[3]{4}\ d_0 \tag{4.13}$$

Substituting $d_0 = d/\sqrt[3]{4}$ into Equation 4.8 gives

$$N_\gamma = \frac{4W}{\pi\gamma d^3 \tan^3(\beta/2)} \tag{4.14}$$

For the conditions adopted in the present study with $W = 60.5$ g and $\beta = 30°$, the following expression is obtained.

$$N_\gamma = \frac{4004.13}{\gamma d^3} \tag{4.15}$$

where d is in cm and γ is in g/cc.

With values of d from Figure 4.4 for Standard sand and Figure 4.6 for Ganga sand, values of N_γ were computed using Equation 4.15 at different void ratios. The corresponding values of ϕ' were estimated using the relationship $\phi' = \tan^{-1}(e_0(\tan\phi_{cv})/e)$, where e_0 is the void ratio corresponding to a relative density of 0.1, which is assumed to be approximately equal to e_{cr} corresponding to the extremely low operating stress levels in the fall cone test.

The results for Standard sand are presented in Figure 4.15, where values of N_γ computed from the results of fall cone tests are plotted against ϕ' estimated according to the procedure outlined above. The figure also depicts a theoretical relationship between N_γ and ϕ' for a rough, shallow foundation having the shape of a right circular cone with an apex angle of $\beta = 30°$, based on results presented by Meyerhof (1961). It may be seen that the values computed from the results of fall cone tests are in close agreement with the values predicted by Meyerhof (1961). The figure also compares the values of N_γ from plate load tests presented by Shahu (1988) with the values predicted by Meyerhof (1961) for a shallow, rough conical foundation with an apex angle of 180° (which would correspond to a circular plate). In this case also there is an excellent agreement between predicted and measured values. Thus results of fall cone tests from the present study and plate load tests conducted by Shahu (1988) confirm the validity of the relationships between N_γ and ϕ' in the case of Standard sand which is a wellrounded sand.

A similar comparison for Ganga sand which is an angular sand is presented in Figure 4.16. In this case the agreement between the results of fall cone and plate load tests and the corresponding theoretical relationships of Meyerhof (1961) is very poor. The figure also depicts the results of plate load tests on Chattahoochee River sand (which has a remarkable resemblance to Ganga sand in nature and behaviour) from Vesic (1963), showing a pronounced deviation from the trend predicted by Meyerhof (1961). Thus values of N_γ from both fall cone and plate load tests for Ganga sand and plate load tests for Chattahoochee River sand do not agree with the corresponding

theoretical predictions. This discrepancy is presumed to be primarily due to the fact that these sands are angular in nature.

Hence it appears that while in the case of rounded sands the theoretical relationships between N_γ and ϕ' seem to be valid, such relationships may not be very accurate in the case of angular sands at very low levels of confining stress.

4.1.7 Morphology and Undrained Shear Behaviour

As the initial state of a sand which is represented by void ratio and effective confining stress determines to a large extent its behaviour during undrained shear, to highlight the effects of particle morphology comparisons have to be made at the same state. Since relative density fails to account for the extremely important influence of confining stress, it is not a satisfactory state variable. Ishihara (1993) points out that while the state parameter (ψ) proposed by Been and Jefferies (1985) is useful for quantifying the behaviour of medium to dense sands under relatively high confining stress, its use is less suitable for studying the behaviour of loose sands at low stress levels.

To overcome some of the limitations of the state parameter, Ishihara (1993) proposed a new state variable called the state index (I_s) which may be defined as $I_s = (e_r - e)/(e_r - e_s)$. Here e is the initial value of the void ratio corresponding to a mean effective principal stress of $P' = (\sigma_1' + 2\sigma_3')/3$. e_r is a reference void ratio which is equal to e_0 for values of confining stress $P' < P'_{ms}$, where e_0 is a threshold void ratio differentiating conditions of zero and non-zero residual strengths and is determined as the void ratio on the steady state line corresponding to $P' = 0$. P'_{ms} is a critical stress obtained as the intersection of the $e = e_0$ line with the isotropic consolidation line (ICL) of the loosest possible sample obtained by moist-placement (wet tamping). For $P' > P'_{ms}$ e_r is the void ratio on the ICL corresponding to the same value of P'. The other reference void ratio e_s is the void ratio on the quasi-steady state line at the same value of P', where the quasi-steady state refers to the transient state corresponding to minimum shear stress and is a particular case of phase transformation where a temporary drop in shear stress takes

place over a limited range of shear strains.

In the present work the effect of particle angularity on the undrained shear behaviour of sands is examined from three angles, using published data for three sands with different morphological characteristics. Firstly, the effect of particle shape on the nature of the steady state lines and isotropic consolidation lines is illustrated. Secondly, the range of initial states corresponding to each different mode of behaviour during undrained shearing of saturated sands, are demonstrated to be significantly influenced by particle angularity. Lastly, the effect of angularity on certain relationships between some parameters characterizing undrained shearing behaviour is explored. The undrained shear behaviour of three quartz sands namely, Toyoura sand (TS), Mine Tailings (MT), and Banding #6 Sand (B6) is studied. The data for TS is from Ishihara (1993), and for MT and B6 from Castro et al. (1982). The available information on the particle size distribution, grain shape and limiting void ratios for these sands is summarized in Table 4.2 .

Table 4.2: Characteristics of Sands Studied

Sand	D_{50} mm	C_u	Fines %	Grain Shape	e_{max}	e_{min}
Toyoura	0.17	1.7	0	Subangular	0.977	0.597
Banding #6	0.157	1.7	0.2	Subrounded	0.82	0.52
Mine Tailings	0.256	2.5	7	Angular	1.08	0.68

Characteristics of ICL and SSL

The influence of of particle angularity on the shape of steady state lines has been clearly brought out by Castro and Poulos (1977), Castro et al. (1982) and Poulos et al. (1985). To illustrate the effect of morphology, the isotropic consolidation line (ICL) and the steady state line (SSL) for TS, MT and B6, from tests performed on specimens prepared by moist placement are shown in Figure 4.17. It may be noted that the influence of grain morphology is

reflected in the shapes of both ICL and SSL, with the change in void ratio for a given change in P', increasing as angularity increases.

State Index and Modes of Undrained Shear Behaviour

An examination of the extensive test data reported for TS (Ishihara 1993) and MT and B6 (Castro et al. 1982) shows that, depending on the initial state of the sand in terms of void ratio and P', four distinct modes of undrained shear behaviour are possible. The influence of particle angularity on undrained shear behaviour of saturated sands is most convincingly demonstrated by examining its influence on the range of initial states for which each type of behaviour is indicated. For this purpose, using the data reported for the three sands the I_s values were calculated. But instead of using the quasi-steady state as recommended by Ishihara (1993), the steady state was used as a reference state since data on quasi-steady state was not available for MT and B6. By comparing the initial states represented by I_s and P' with the behaviour exhibited during undrained shear, the limiting values of I_s for each mode of behaviour were determined.

The four different types of behaviour are depicted in Figures 4.18. 4.19 and 4.20 for MT, TS and B6 respectively. Each figure shows the limiting I_s values demarcating the range of initial states for which each type of behaviour is exhibited. The salient features of each type of behaviour in terms of the stress-strain curve ($q = \sigma_1 - \sigma_3$ versus axial strain ϵ_a) and the effective stress path (q versus P') are also illustrated. The ranges of initial states in terms of I_s and P' values for each type of behaviour are summarized in Table 4.3.

The characteristic features of each type of behaviour are:

Type 1 $(\sigma_1 - \sigma_3)_{max}$ mobilized at small strains; zero residual undrained shear strength; mobilized effective angle of shearing resistance less than the steady state friction angle — characterizes meta-stable flow failure.

Type 2 After attaining $(\sigma_1 - \sigma_3)_{max}$ shearing resistance decreases monotonically to reach steady state at large strains. Finite residual undrained shear strength.

Table 4.3: Types of undrained shear behaviour and corresponding initial states

Sand	e_0	P'_{ms} MPa	Types of Behaviour and Initial States				
			Type 1		Type 2	Type 3	Type 4
			I_s	P'	I_s	I_s	I_s
TS	0.93	1.5	≤ 0	$< P'_{ms}$	0–0.2	0.2–1.0	≥ 1.0
MT	1.065	0.2	≤ 0	$< P'_{ms}$	0–0.3	0.3–1.0	≥ 1.0
B6	0.81	0.64	≤ 0	$< P'_{ms}$	0–0.55	0.55–1.0	≥ 1.0

Type 3 $(\sigma_1 - \sigma_3)$ after attaining the peak decreases to a minimum and then increases again and approaches steady state at large strains. Characterizes quasi-steady state (QSS) behaviour.

Type 4 Dilatant response throughout shearing, except possibly at small strains where behaviour may be contractive.

It may be noted that the range of I_s values for which Type 1 and 2 modes are indicated, is significantly influenced by grain morphology. The value of e_0 which together with the location of the ICL determines the magnitude of the critical stress P'_{ms} separating Type 1 and 2 modes, increases as angularity increases (see Table 4.3). The range of initial e and P' values for which Type 1 behaviour is indicated is least for the angular sand, whereas it is more for the subangular and the subrounded sands. This is consistent with the observation of Terzaghi and Peck (1948) that *the most unstable sands so far encountered consist chiefly of rounded grains.* It may also be seen that in the case of angular and subangular sands, Type 2 behaviour is indicated for I_s between 0 and 0.2–0.3, whereas the subrounded sand exhibits the same behaviour over a wider range of I_s values (0–0.55).

Grain Morphology and Some Undrained Shear Parameters Considered by Ishihara

Ishihara (1993) in his studies on TS, related the initial effective confining stress (P'_c) to the value of P' corresponding to the maximum value of q (P'_p).

He also examined the influence of I_s on the initial stress ratio (r_c) defined as P'_c/P'_s where P'_s is the value of P' at steady state, the peak undrained shear strength s_p normalized with respect to the initial confining stress σ'_0 and the residual undrained shear strength s_{us} normalized with respect to σ'_0. In the present study similar relationships were obtained for MT and B6 from the data reported by Castro et al. (1982). A comparison of these relationships for the three sands reveals the effect of grain angularity.

The relationship between P'_p and P'_c for MT and B6 and that proposed for TS by Ishihara is shown in Figure 4.21. Linear relationships with slopes of 0.72, 0.61 and 0.54 for MT, TS and B6 respectively are observed. It may be seen that the slope increases as angularity increases. These relationships are applicable for Type 2 and 3 behaviour described earlier. For TS in the case of Type 1 behaviour, for which the results were kindly made available by Professor Ishihara, the slope is 0.74 compared to 0.61 for Type 2 and 3 behaviour.

The relationship between r_c and I_s is depicted in Figure 4.22 which illustrates the extreme sensitivity of r_c to I_s. It may be observed that for a given value of I_s the subrounded sand shows the maximum value of r_c. which is indicative of very high pore water pressures at steady state and hence a high degree of instability. The angular sand exhibits the least value of r_c, suggesting a lower degree of instability compared to subrounded and subangular sands.

The relationships of s_p/σ'_0 and s_{us}/σ'_0 with I_s are given in Figures4.23 and 4.24 respectively. Here $s_p = (\sigma_1 - \sigma_3)_{max}/2$ and $s_{us} = (q_{ss}/2)\cos\phi'_{ss}$ where q_{ss} is equal to $\sigma'_1 - \sigma'_3$ at steady state. Again linear relationships are observed as suggested by Ishihara and these are valid for Type2 and 3 behaviour only. The observed trends indicate that these relationships are significantly influenced by grain angularity. As Type2 and 3 modes imply a finite value of residual undrained shear strength, it is suggested that Ishihara's trend line for TS may be better represented by the dashed line which stipulates a non-zero value of s_{us} at $I_s = 0$. The values of s_{us}/σ'_0 at $I_s = 0$ are 0.076 for angular (MT), 0.015 for subangular (TS) and 0.0008 for subrounded (B6)

sands.

4.1.8 Morphology and Constrained Modulus

The information available in literature, on the influence of particle morphology on compressibility and constrained modulus was reviewed in Chapter 2. A quantitative evaluation of the effect of particle angularity is most conveniently made in terms of certain parameters of the models for one-dimensional stress-strain behaviour of sands proposed by Janbu (1963, 1985) and Bellotti et al. (1985).

Janbu (1963, 1985) proposed that the constrained modulus (M) of a normally consolidated sand corresponding to a particular relative density and a vertical effective stress of σ'_v may be approximated by

$$M = m(\sigma'_v \sigma_a)^{0.5} \tag{4.16}$$

where σ_a is the atmospheric pressure and m is a dimensionless modulus number which is equal to the value of M/σ_a at $\sigma'_v = \sigma_a$. Based on the results of calibration chamber tests on two sands, Bellotti et al. (1985) present a more general equation

$$M = m_0(\sigma'_v/\sigma_a)^{m_1} \exp(m_2 I_D)\sigma_a \tag{4.17}$$

where m_0 is a dimensionless modulus number which is equal to the value of M/σ_a corresponding to $\sigma'_v = \sigma_a$ and $I_D = 0$ and m_1 and m_2 are empirical constants which account for the variation of M with stress and relative density respectively. Rahim (1989) discusses the influence of particle roundness on the coefficients m_0, m_1 and m_2. In the present work, results presented by Rahim(1989) together with additional information obtained from the data reported by Castro (1969), Castro et al. (1982) and Hryciw and Thomann (1993), were used to quantitatively evaluate the influence of roundness on the coefficients m, m_0, m_1 and m_2. These model parameters were evaluated using the one-dimensional compression data presented by Castro (1969) and Castro et al. (1982) for the sands studied by them. Hryciw and Thomann (1993) report values of the dimensionless stiffness coefficient S_{1Dmax}, which

is a parameter in the model for one-dimensional strain proposed by Hardin (1987). Hardin's model in the low stress range may be expressed as

$$\frac{1}{e} = \frac{1}{e_i} + \frac{1}{S_{1Dmax}}(\frac{\sigma'_v}{\sigma_a})^p \tag{4.18}$$

where e and e_i are void ratios at a vertical stress of σ'_v and zero respectively and p is an empirical constant for which a value of 0.5 was recommended by Hardin. By differentiating both sides of Equation 4.18 with respect to σ'_v, the following relationship is obtained.

$$M = \frac{S_{1Dmax}}{p}\frac{1+e_i}{e^2}(\frac{\sigma'_v}{\sigma_a})^{1-p}\sigma_a \tag{4.19}$$

From the test results presented by Hryciw and Thomann (1993), M was computed using Equation 4.19 and values of the parameters m, m_0, m_1 and m_2 were obtained.

The effect of particle angularity on constrained modulus is illustrated in Figure 4.25, where the variation of modulus number m (which is equal to the value of M/σ_a at $\sigma'_v = \sigma_a$) with relative density is presented. The curves shown in the figure represent the relationships proposed by Rahim (1989) for these sands. It should be noted that Ganga, Kalpi and Mine Tailings are angular, Hokksund subangular, Ticino subrounded, Ottawa rounded and Standard wellrounded. Additional data for some more sands, computed from the results presented by Castro (1969), Castro et al. (1982) and Hryciw and Thomann (1993) are also presented. The following general trends may be observed. At any given relative density rounded sands have a higher value of m than angular sands. The variation of m with relative density appears to be more pronounced for rounded sands in comparison to angular sands.

The variation of m_0, m_1 and m_2 with roundness is depicted in Figure 4.26. In spite of considerable scatter, the following trends may be noted. Roundness has a significant influence on m_0 with change in roundness alone causing a variation in m_0 by a factor of about 3–4. m_1 appears to be independent of roundness. Janbu (1985), Hardin (1987) and Hryciw and Thomann (1993) suggest a value of 0.5 for m_1 and this value appears to be a good average for the data points from Rahim (1989) shown in the figure. The scatter in the

values of m_1 may be due to a stress level effect and for all practical purposes m_1 may be taken to be 0.5. In many cases m_2 was evaluated on the basis of the limited data available and perhaps this could be a possible reason for the large scatter in the values of m_2. Nevertheless, m_2 appears to increase with roundness in the angular and subangular range and then remains more or less constant.

4.1.9 Morphology and Secondary Compression

The time dependent deformation occurring under constant vertical effective stress is termed as secondary compression to distinguish it from the deformation resulting from a change in vertical effective stress. Though the magnitude of secondary compression of sands is considerably less than that for clays, some researchers consider this effect to be significant for accurate prediction of settlements of foundations on sands (Schmertmann 1970, Burland and Burbidge 1985). Another problem of great practical relevance is the phenomenon of ageing, in which a great deal of interest has been generated in the recent years. Secondary compression is considered to be an important mechanism of ageing of sands (Mesri et al. 1990, Schmertmann 1991). For a better understanding of the ageing phenomena and development of methods for prediction of resulting improvements in behaviour, a thorough understanding of the nature of secondary compression characteristics of sands is essential.

In the present work oedometer tests were carried out on four different materials—Steel bearing balls, Standard sand, Ganga sand and Crushed rock, according to the test procedures described in Chapter 3. Typical results showing the increase in vertical strain with time, induced by a change in the vertical stress from 1100 to 1600 kPa are presented in Figure 4.27 for Steel bearing balls and Standard sand, Figure 4.28 for Ganga sand and Figure 4.29 for Crushed rock. In the case of Steel bearing balls and Standard sand, the relationship between strain increment and logarithm of time is linear right from the beginning. But for Ganga sand and Crushed rock, linearity is observed only beyond a time ranging from 1 to 2 minutes. For time periods

less than this, rate of deformation is found to be higher. Several factors like relative density, stress increment ratio and duration of each increment were found to affect the form of strain increment versus logarithm of time relationships. But for the data presented in Figures 4.27, 4.28 and 4.29, stress increment ratio and duration of previous increments were same and in the case of Standard and Ganga sands the relative densities were also identical. Therefore the difference in behaviour is believed to be primarily due to differences in the nature of the sand, especially with regard to particle morphology. The larger rates of deformation during early stages in the case of angular materials like Ganga sand and Crushed rock could be in part due to the contribution from particle crushing and modification.

For a better comparison of the behaviour of different materials the data shown in Figures 4.27, 4.28 and 4.29 are presented together in Figure 4.30. The difference between Ganga and Standard sands having practically identical relative densities is thought to be primarily due to differences in particle morphology. This figure also illustrates the fact that for materials exhibiting large initial compression, the time dependent component also would be larger. This is in accordance with the concept of C_α/C_c being a constant, postulated by Mesri and his co-workers (Mesri and Godlewski 1977, Mesri 1987, Mesri and Castro 1987).

Based on extensive research on the behaviour of a wide variety of soils, Mesri and his co-workers arrived at the conclusion that for a soil at any value of vertical stress and time, the ratio C_α/C_c is a constant. Here $C_\alpha = -\Delta e/\Delta \log t$ is the secondary compression index and $C_c = -\Delta e/\Delta \log \sigma'_v$ is the compression index. For granular materials a value of $C_\alpha/C_c = 0.02 \pm 0.01$ is recommended by Terzaghi et al. (1996). From the results of oedometer tests conducted in the course of the present study, values C_α and C_c were computed for the four materials studied. It should however be noted that the various shortcomings of conventional oedometer tests may introduce some amount of uncertainty into the measurements (Cornforth 1974, Mesri et al. 1990). Apparatus compliance, side friction, bedding error, temperature fluctuations, vibrations and other mechanical disturbances have been suggested

as potential sources of error. Though all possible precautions were taken to minimize some of these errors, many factors were beyond control and hence it is possible that the measurements were subject to some error. In spite of these uncertainties, the following inferences can be made from the results presented in Figure 4.31.

For all the four materials, computed values of C_α/C_c lie between 0.01 and 0.02 and hence are within the range suggested by Terzaghi et al. (1996). The following values of C_α/C_c of 0.015, 0.01, 0.012 and 0.014 may be considered as representative averages for Steel bearing balls, Standard sand, Ganga sand and Crushed rock respectively. Except in the case of Steel bearing balls, there appears to be a trend, though admittedly small, of C_α/C_c increasing with increasing particle angularity. Considering the fact that in general C_α/C_c increases with increasing compressibility (Terzaghi et al. 1996 recommend values ranging from 0.02 ± 0.01 for granular soil including rock fills to 0.06 ± 0.01 for peat and muskeg), this is intuitively admissible. If this is true, then Steel bearing balls should have a value of C_α/C_c which is less than or equal to that of Standard sand. It is thought that the discrepancy in the data shown in Figure 4.31 could be due to the effect of side friction. It is likely that in the case of steel bearing balls side friction is minimum and the measured value of C_α/C_c is very close to the true value. If a corrected value of 0.015 instead of the measured value of 0.01 is assumed to be applicable for Standard sand and the measured values for Ganga sand and Crushed rock are adjusted by the same factor, the following values of 0.015, 0.015, 0.018 and 0.021 are indicated for Steel bearing balls, Standard sand, Ganga sand and Crushed rock respectively. For all practical purposes, a value of 0.02 ± 0.01 could be adopted for granular materials.

4.1.10 Morphology and Behaviour in 1-D Cyclic Loading

It is well-known that overconsolidation and secondary compression result in significant improvements in the stress-deformation behaviour of sands, due

to the effects of strain hardening and increase in horizontal effective stress. Cyclic straining which in nature can be induced by seismic activity, could also lead to similar changes in behaviour. Therefore an investigation of the behaviour of sands in one-dimensional cyclic loading, in relation to particle morphology could be of considerable interest. In the present study, materials having different morphological characteristics, were subjected to repeated loading and unloading cycles in the oedometer. Tests were conducted wherein the vertical stress was cyclically varied between 100 and 400 kPa and between 400 and 1600 kPa.

The development of additional strains during cyclic loading between 400 and 1600 kPa is illustrated in Figure 4.32, where the cumulative increase in strain is plotted against the logarithm of the number of cycles of loading. Strains induced in Steel bearing balls are quite small, even though the sample had a void ratio of 0.69 which is extremely close to the e_{max} of 0.70. Strains developed in Ganga sand are larger than in Standard sand, although both have the same relative density. This difference may be considered to be primarily due to the differences in angularity. Though the accumulation of strains is maximum in the case of Crushed rock, a proper comparison is difficult since the relative density is not known in this case. Similar data for Standard and Ganga sands with cyclic loading between 100 and 400 kPa is presented in Figure 4.33. Here also it can be seen that strains are induced at a faster rate in the angular Ganga sand in comparison to the wellrounded Standard sand, even though the relative density is same in both cases. The general picture that emerges from these results is that compressible sands tend to accumulate more strains during one-dimensional cyclic loading. This suggests a similarity between behaviour in one-dimensional cyclic straining and secondary compression.

Differences between Standard and Ganga sands, presumably due to their dissimilar morphological characteristics, are also suggested by the nature of the hysteresis loops observed in one-dimensional cyclic loading. The hysteresis loops for the first and hundredth cycles of loading between 100 and 400 kPa, for Standard and Ganga sands having practically identical relative

densities are depicted in Figure 4.34. Similar data for loading between 400 and 1600 kPa is presented in Figure 4.35. These figures suggest that at the end of about 100 cycles the loops practically close. Two differences between the stabilized loops of Ganga and Standard sands may be observed. Firstly, the size of the loop for Standard sand is considerably smaller than that of Ganga sand. The area of the hysteresis loop represents the energy dissipated during the deformation process and this is seen to be more for angular sands in comparison to rounded sands. In the case of metals hysteresis is caused by the imperfections of the crystal structure, but in the case of granular materials this may be considered to arise from the particle arrangement. Secondly, the average slope of the stabilized loop is found to be larger for Ganga sand. Since the reciprocal of this slope may be considered to be representative of an average elastic modulus, a higher value of modulus is indicated for rounded sands which is consistent with the expected trend.

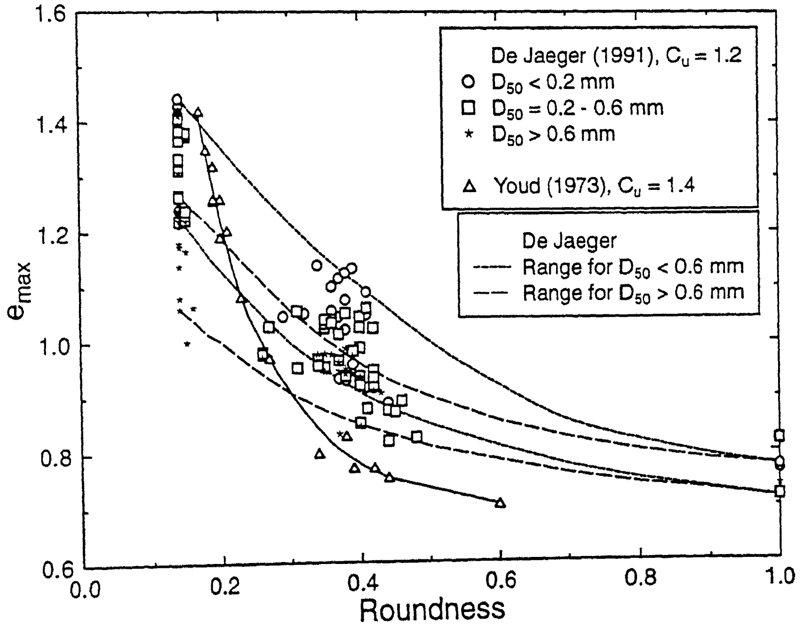

Figure 4.1: The relationship between e_{max} and roundness for sand fractions with the same C_u (data from Youd 1973, De Jaeger 1991).

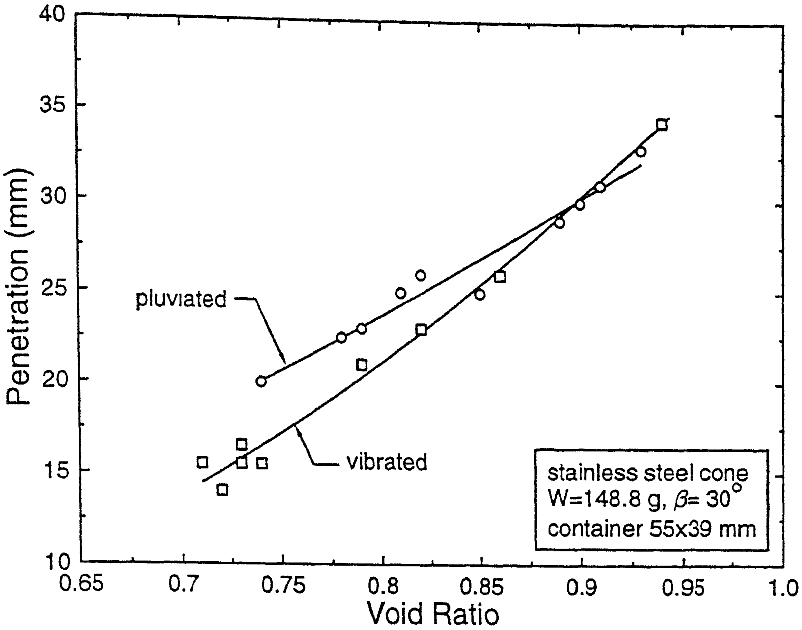

Figure 4.2: The effect of method of specimen preparation—fall cone tests on Ganga sand.

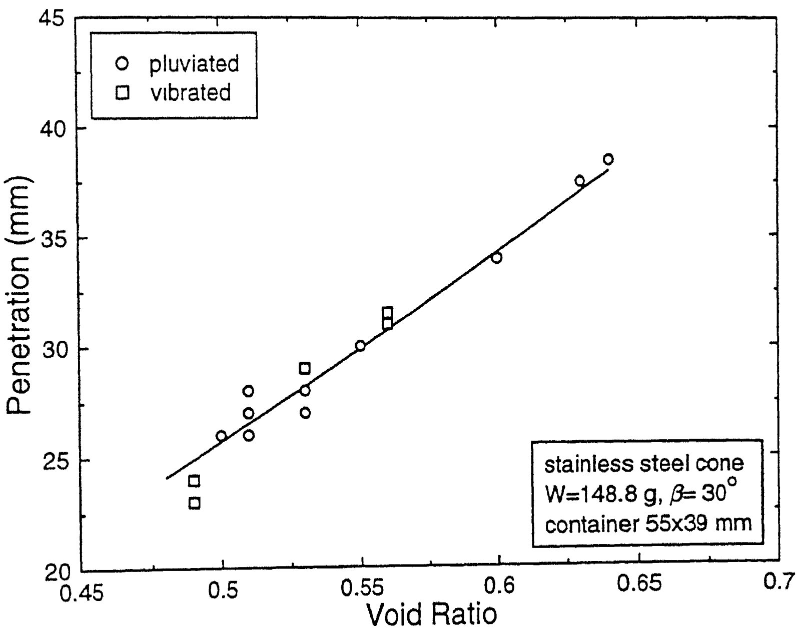

Figure 4.3: The effect of method of specimen preparation—fall cone tests on Standard sand.

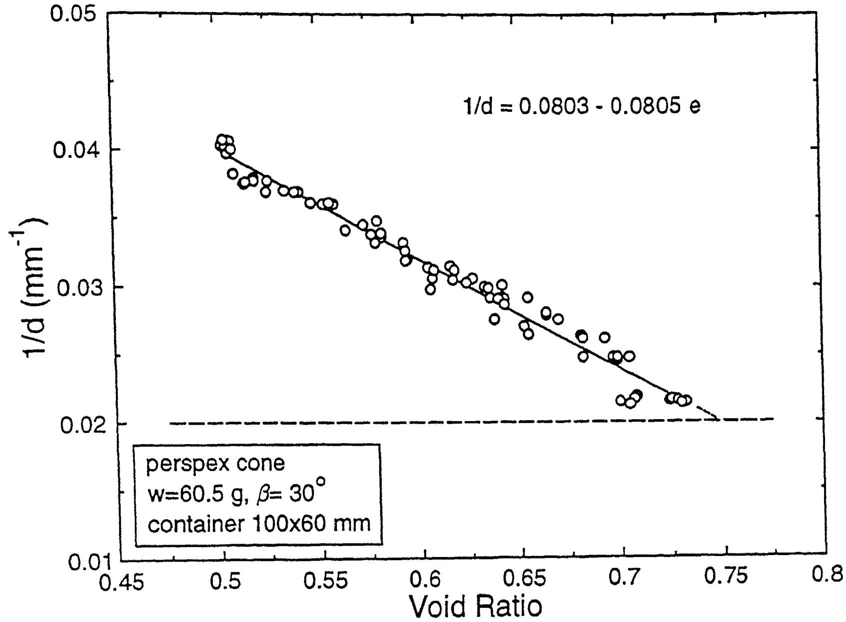

Figure 4.4: Variation of $1/d$ with void ratio for Standard sand.

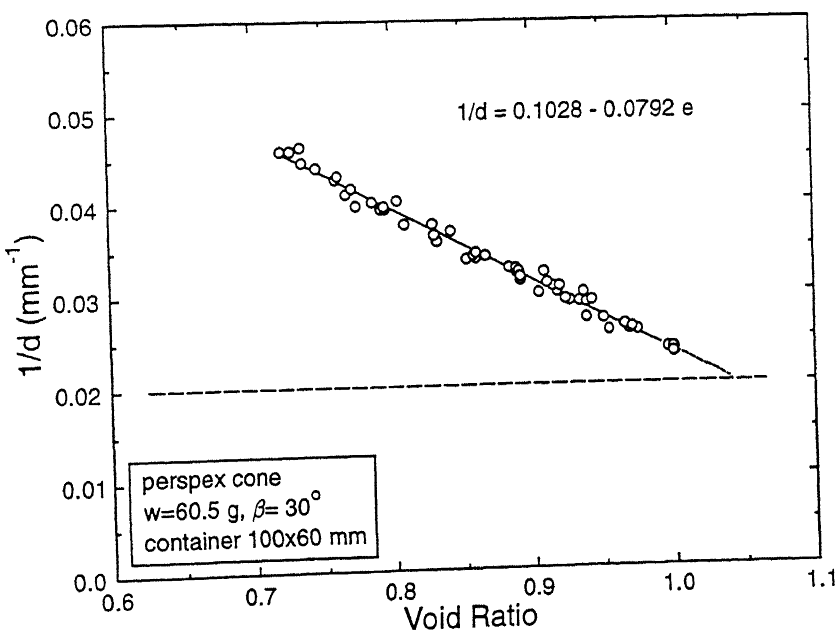

Figure 4.5: Variation of $1/d$ with void ratio for Fine Kalpi sand.

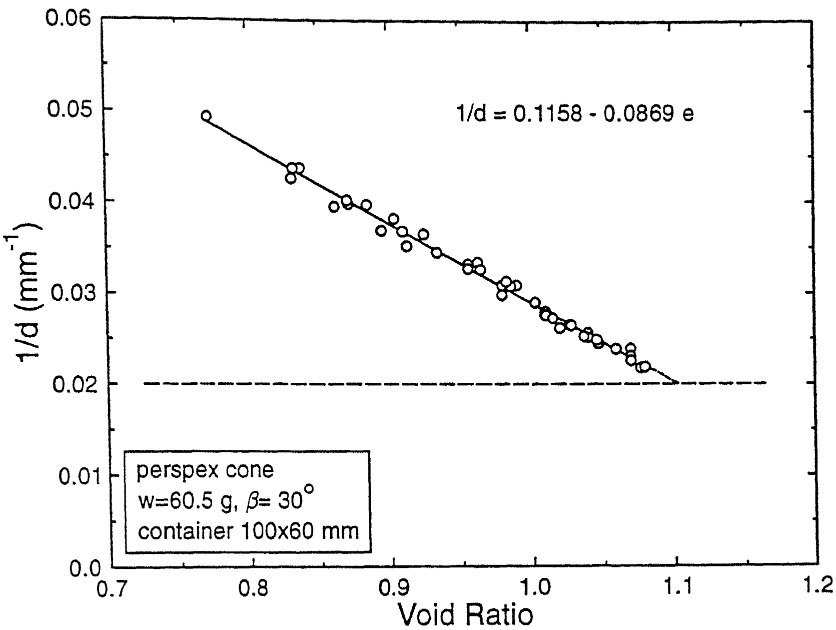

Figure 4.6: Variation of $1/d$ with void ratio for Ganga sand.

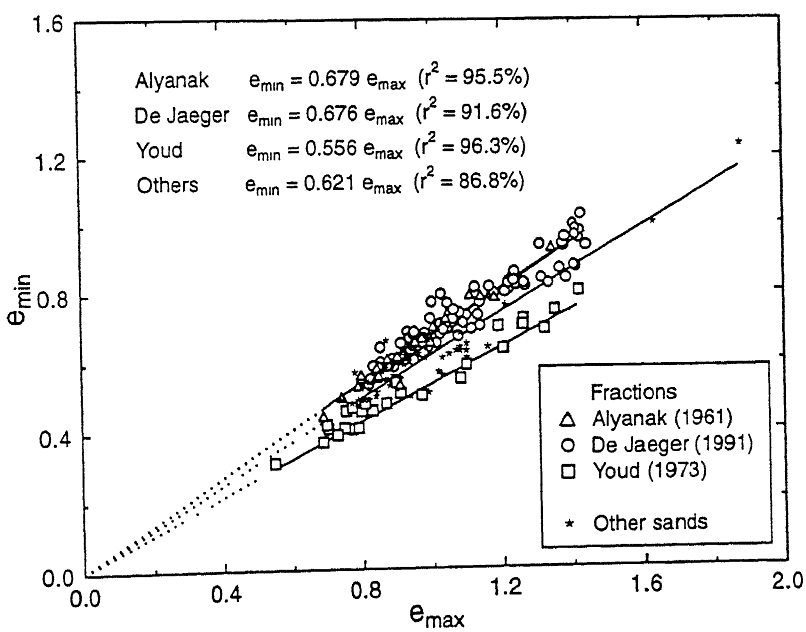

Figure 4.7: The relationship between e_{max} and e_{min} for sand fractions and poorly graded non-micaceous sands (data from Alyanak 1961, Castro 1969, Youd 1973, Broms 1976, Castro et al. 1982, Skempton 1986, Bolton 1986, Been et al. 1987, Rahim 1989 and De Jaeger 1991).

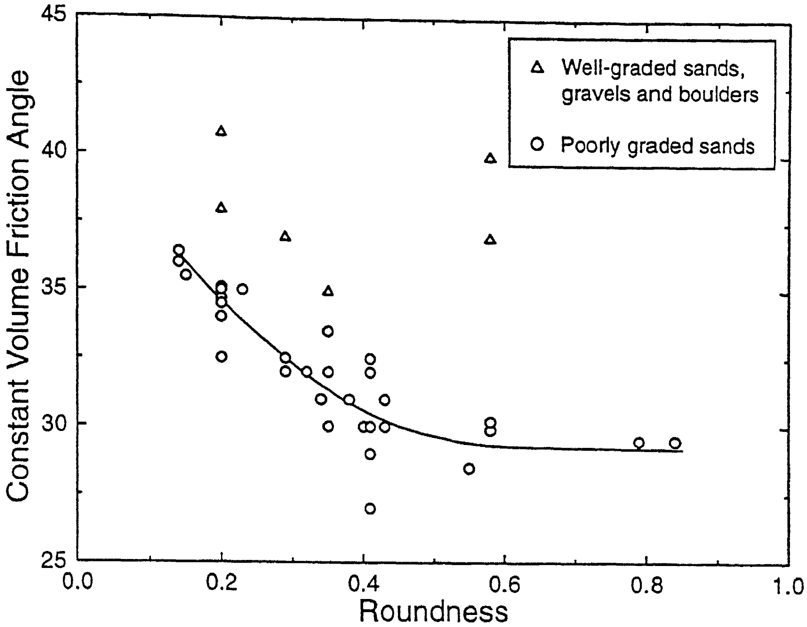

Figure 4.8: The relationship between ϕ_{cv} and roundness for predominantly quartz sands (data from Castro 1969, Castro et al. 1982, Sladen 1985, Been et al. 1987, Negussey et al. 1988, Rahim 1989 and Pandey 1993).

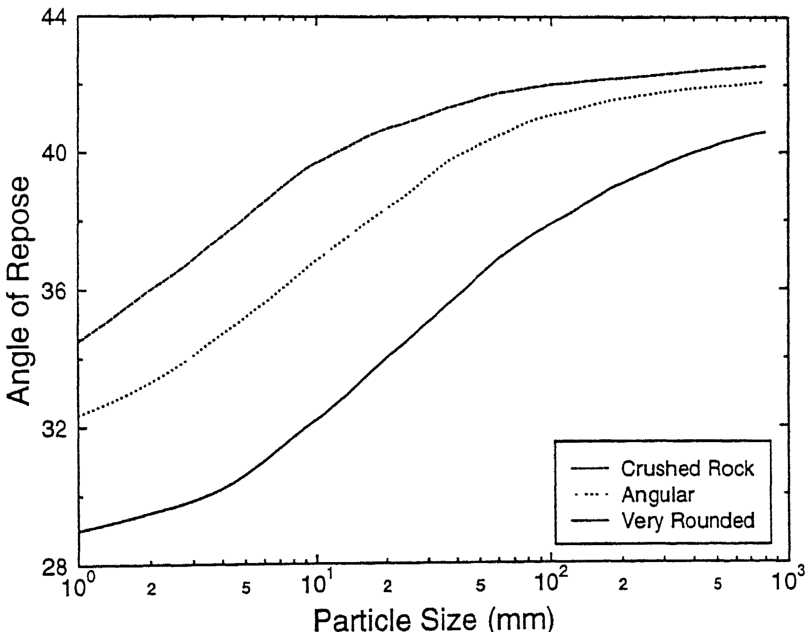

Figure 4.9: The effect of particle size and angularity on angle of repose of rock fills (after Stephenson 1979).

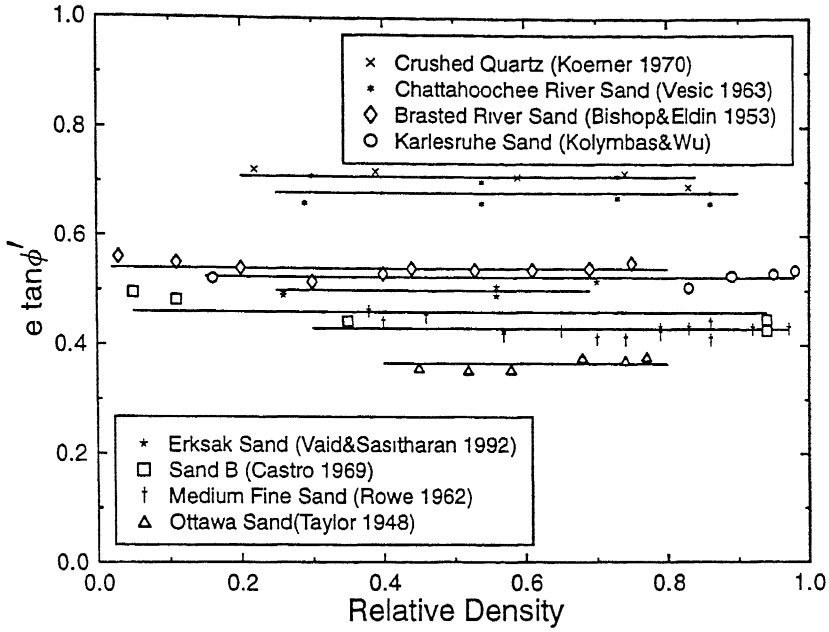

Figure 4.10: $e \tan \phi'$ versus relative density for different sands.

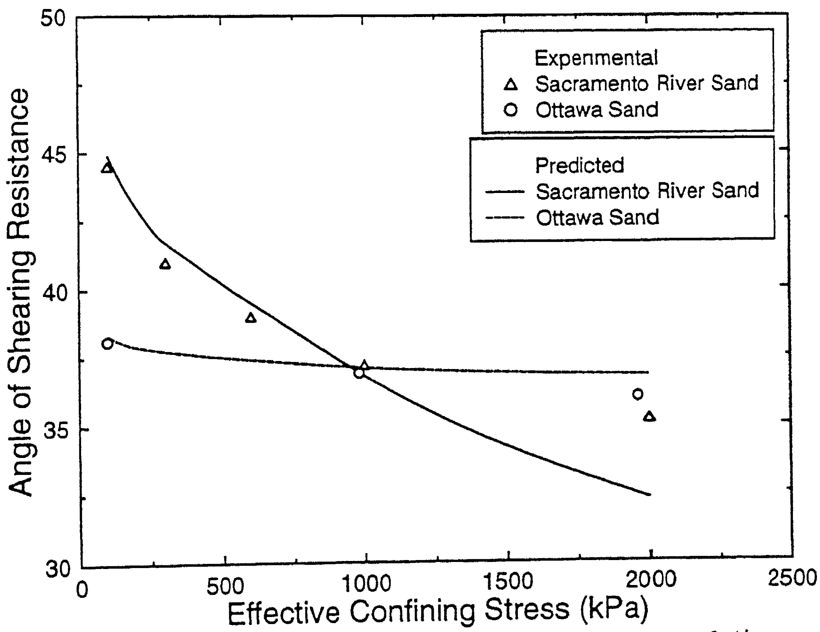

Figure 4.11: Comparison of experimentally determined values of ϕ' reported by Lee and Seed (1967) with those predicted using Equation 4.5.

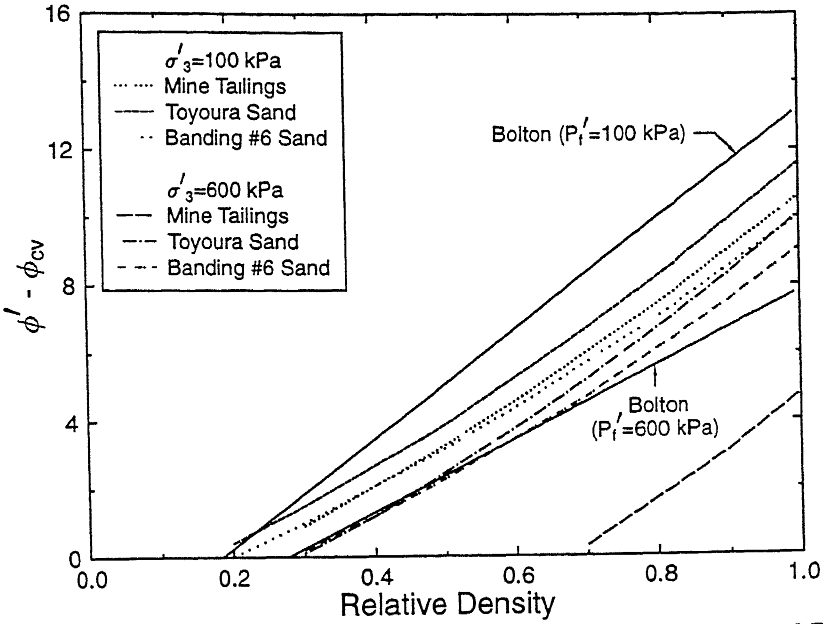

Figure 4.12: Prediction of dilatational component of ϕ'—comparison of Equation 4.5 with Bolton's method (data for MT and B6 from Castro et al. 1982 and for TS from Ishihara 1993).

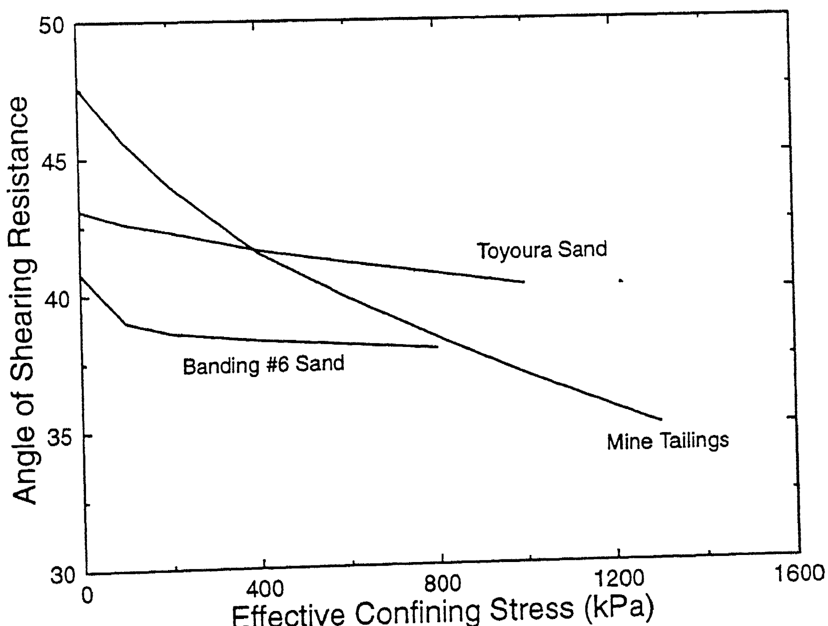

Figure 4.13: Effect of confining stress on ϕ' for sands of different angularity and having $I_D = 1$, as predicted by Equation 4.5 (data for MT and B6 from Castro et al. 1982 and for TS from Ishihara 1993).

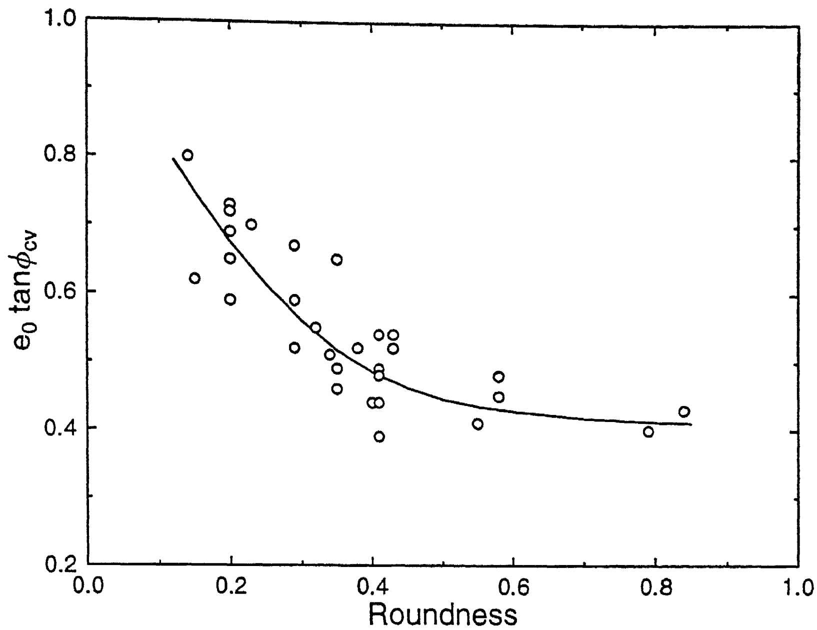

Figure 4.14: The effect of grain angularity on $e_0 \tan \phi_{cv}$ for poorly graded, predominantly quartz sands (data from Castro 1969, Castro et al. 1982, Sladen 1985, Been et al. 1987, Negussey et al. 1988, Rahim 1989 and Pandey 1993).

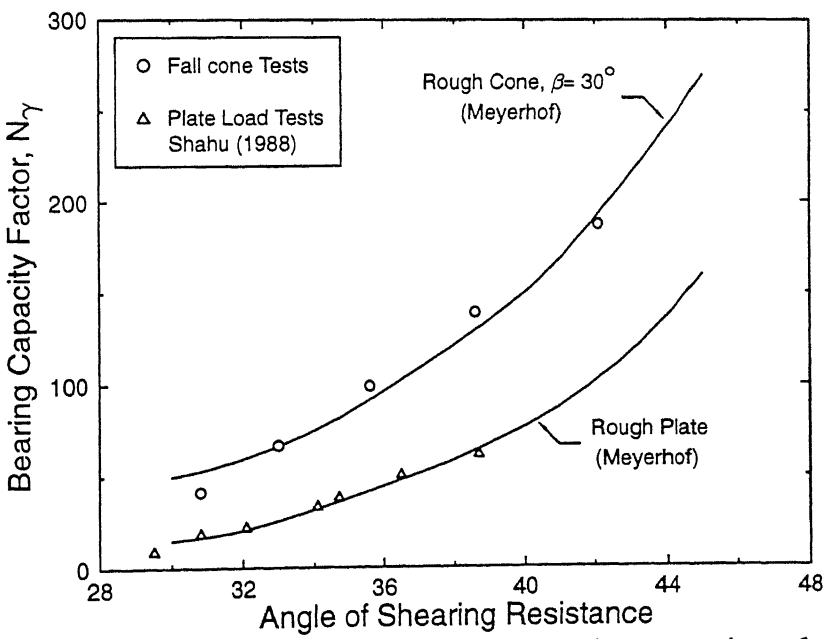

Figure 4.15: Variation of N_γ with ϕ' for Standard sand— comparison of experimental results with Meyerhof's theory.

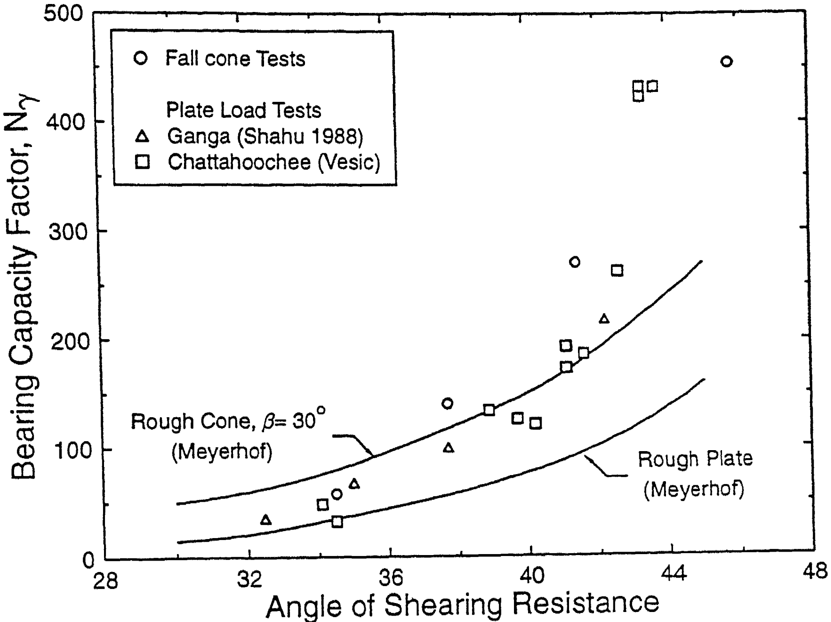

Figure 4.16: Variation of N_γ with ϕ' for Ganga sand— comparison of experimental results with Meyerhof's theory.

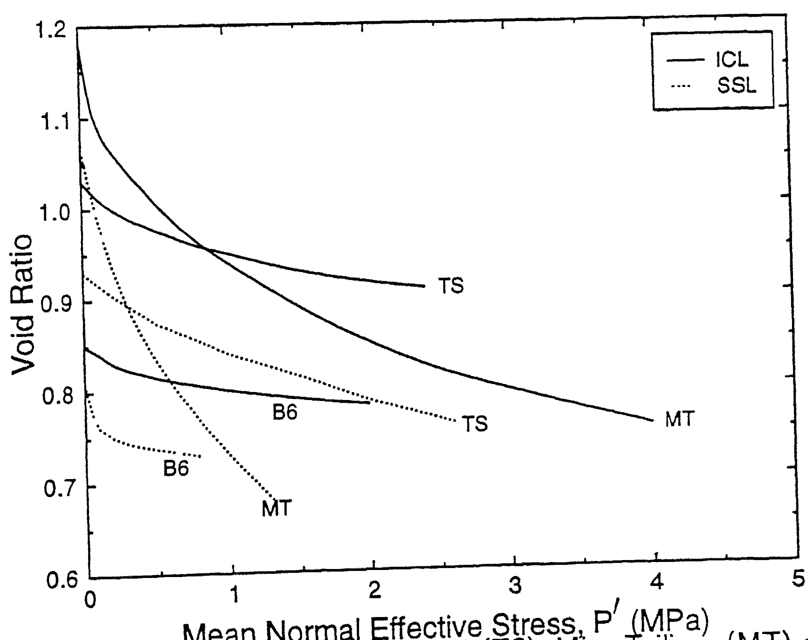

Figure 4.17: ICL and SSL for Toyoura sand (TS), Mine Tailings (MT) and Banding #6 sand (B6) (Data for TS from Ishihara 1993 and for MT and B6 from Castro et al. 1982).

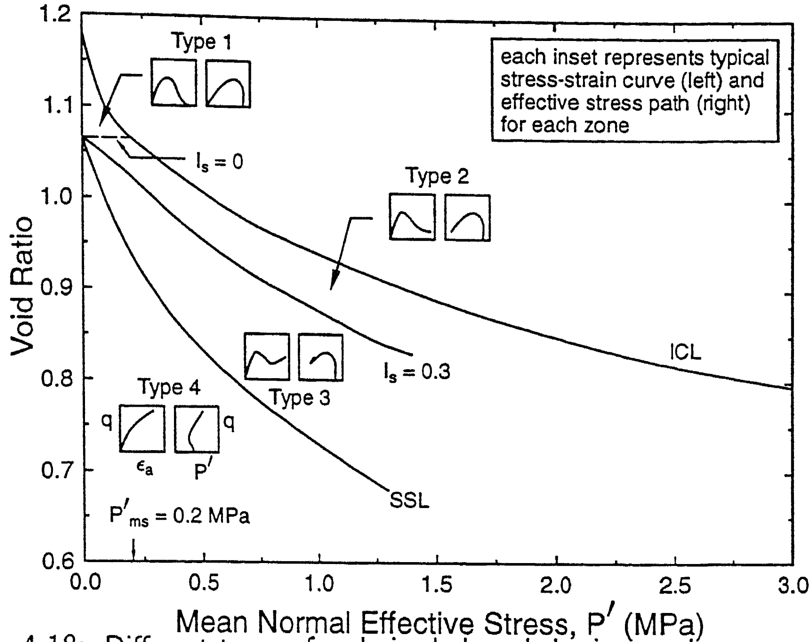

Figure 4.18: Different types of undrained shear behaviour and corresponding range of initial states for Mine Tailings (data from Castro et al. 1982).

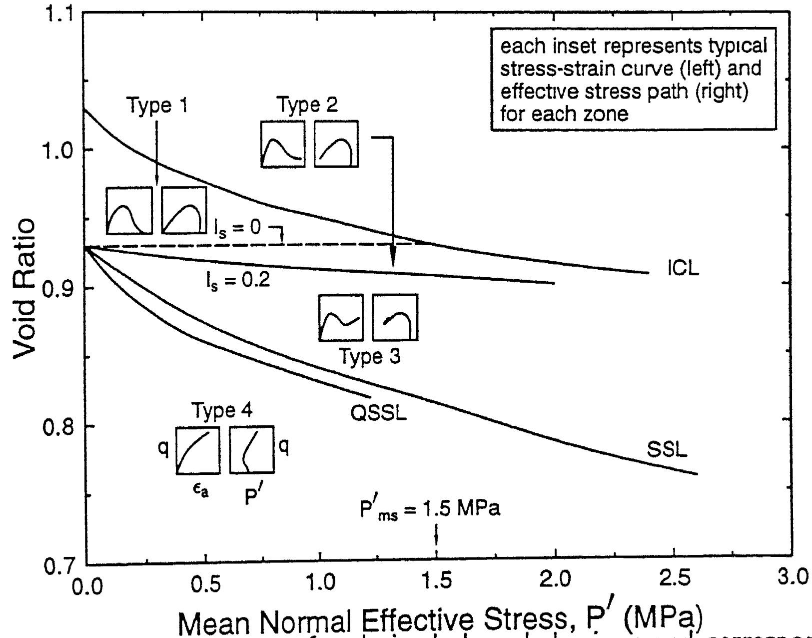

Figure 4.19: Different types of undrained shear behaviour and corresponding range of initial states for Toyoura sand (data from Ishihara 1993).

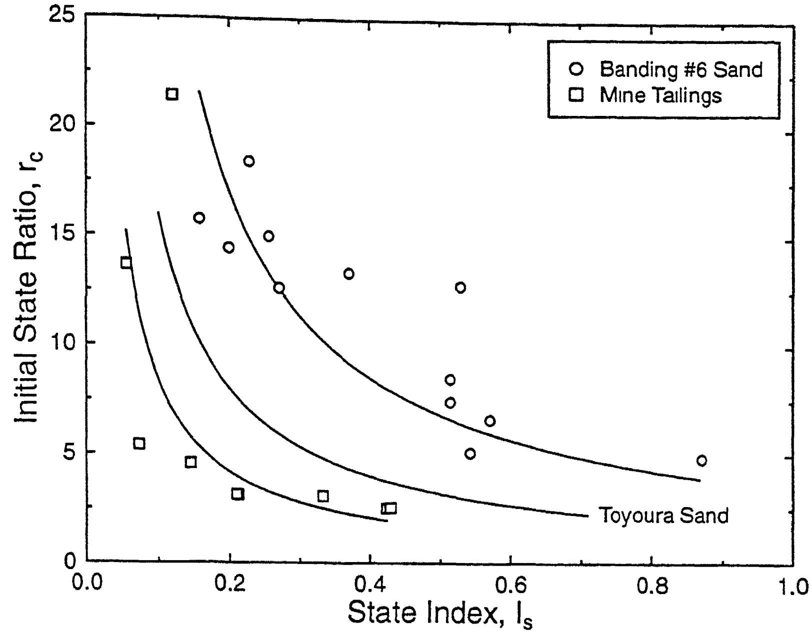

Figure 4.22: r_c versus I_s for B6, TS and MT (data for TS from Ishihara 1993 and for MT and B6 from Castro et al. 1982).

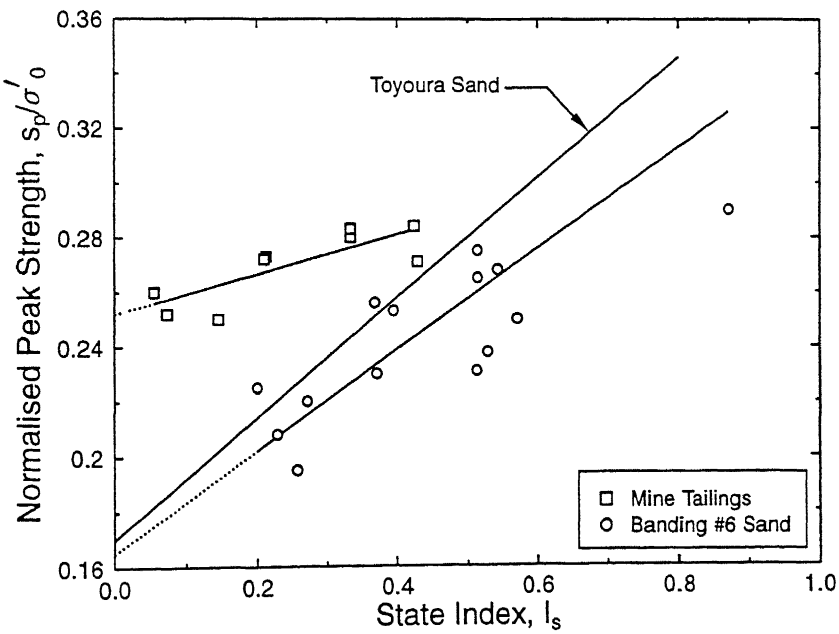

Figure 4.23: s_p/σ'_0 versus I_s for B6, TS and MT (data for TS from Ishihara 1993 and for MT and B6 from Castro et al. 1982).

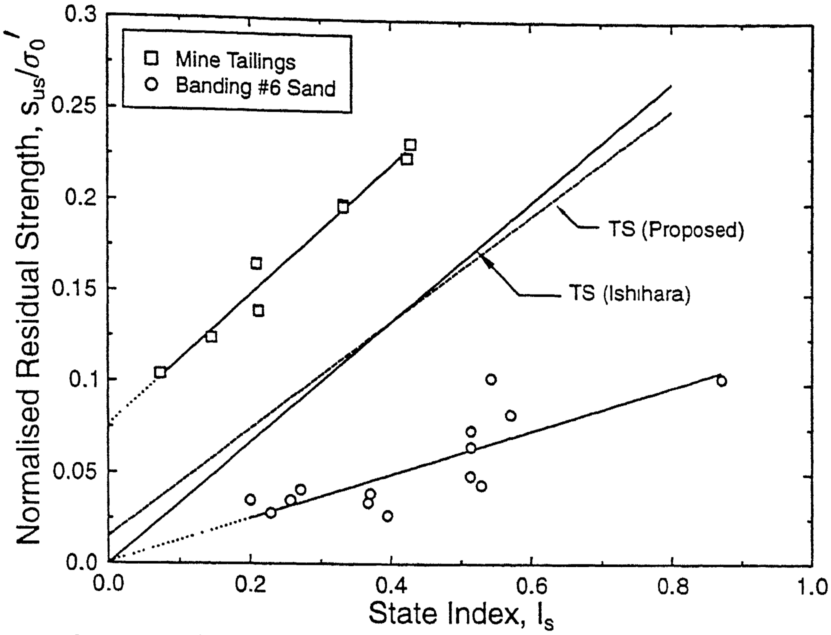

Figure 4.24: s_{us}/σ_0' versus I_s for B6, TS and MT (data for TS from Ishihara 1993 and for MT and B6 from Castro et al. 1982).

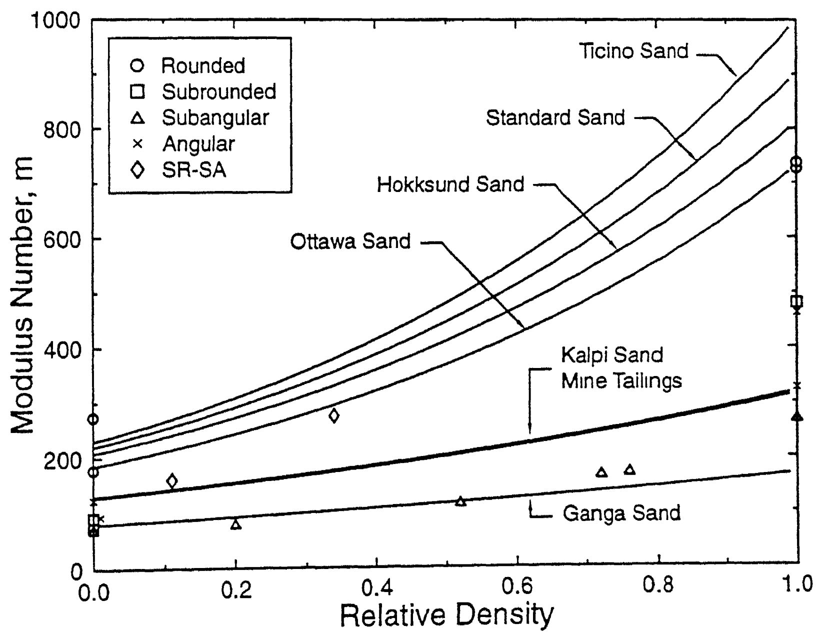

Figure 4.25: Variation of (m) with I_D for sands having different angularity (data from Castro 1969, Castro et al. 1982, Rahim 1989 and Hryciw and Thomann 1993).

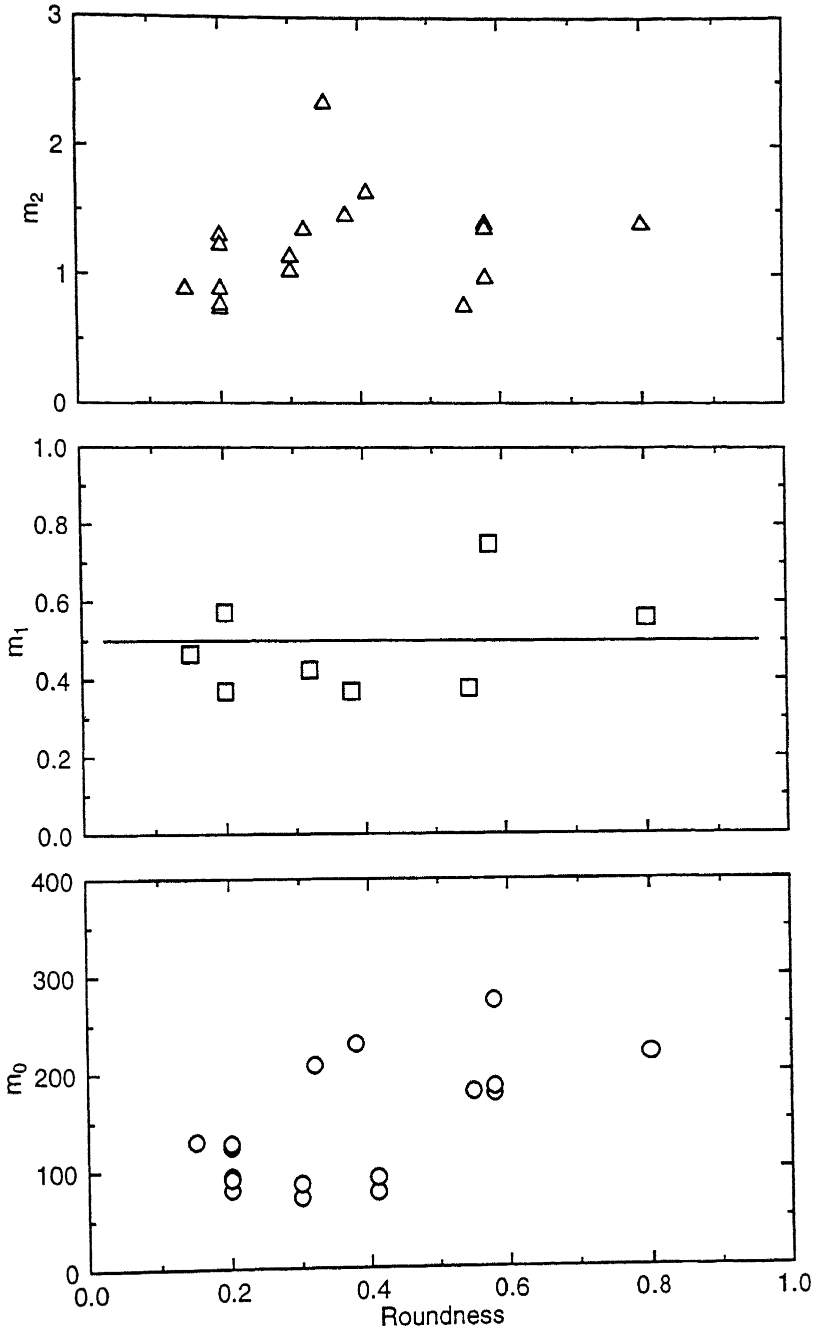

Figure 4.26: Variation of coefficients m_0, m_1 and m_2 with roundness (data from Castro 1969, Castro et al. 1982, Rahim 1989 and Hryciw and Thomann 1993).

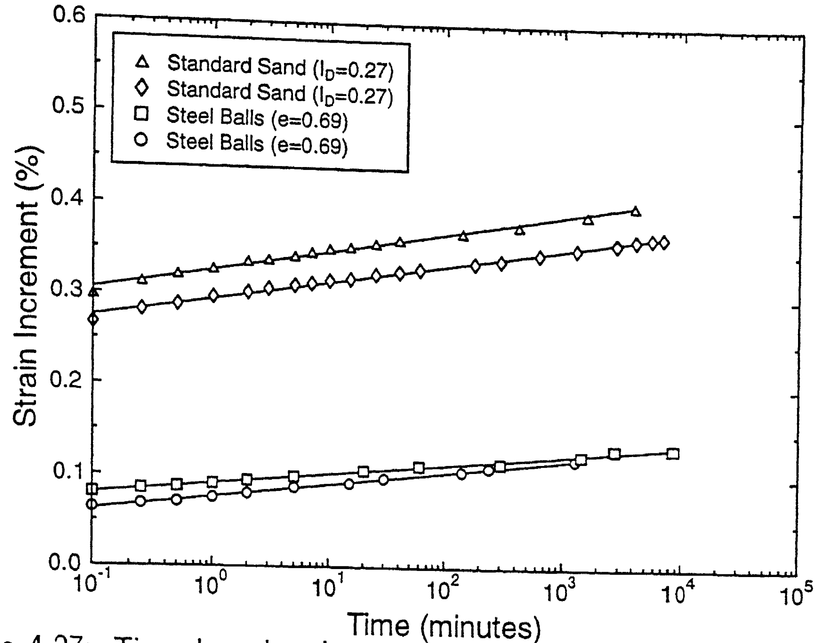

Figure 4.27: Time dependent increase in vertical strain—Standard sand and Steel balls, stress increment 1100–1600 kPa.

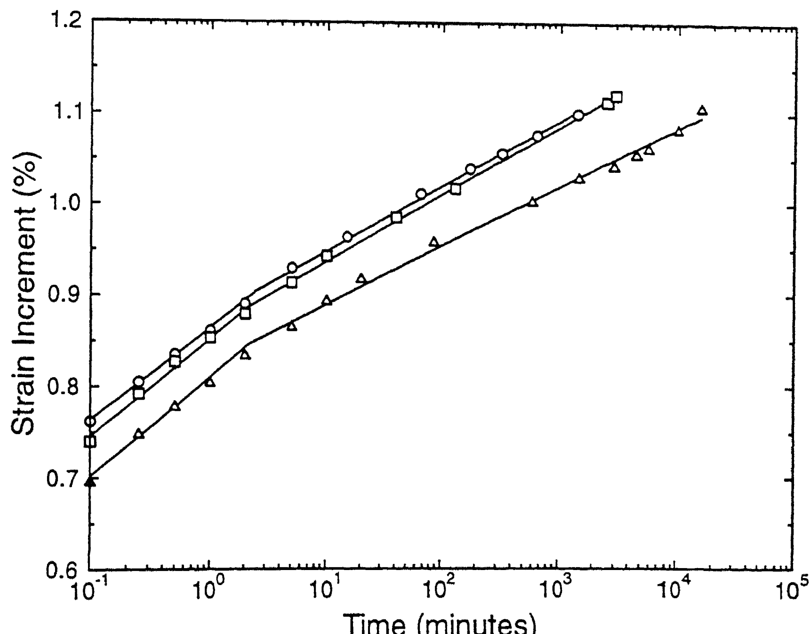

Figure 4.28: Time dependent increase in vertical strain—Ganga sand, I_D = 0.25, stress increment 1100–1600 kPa.

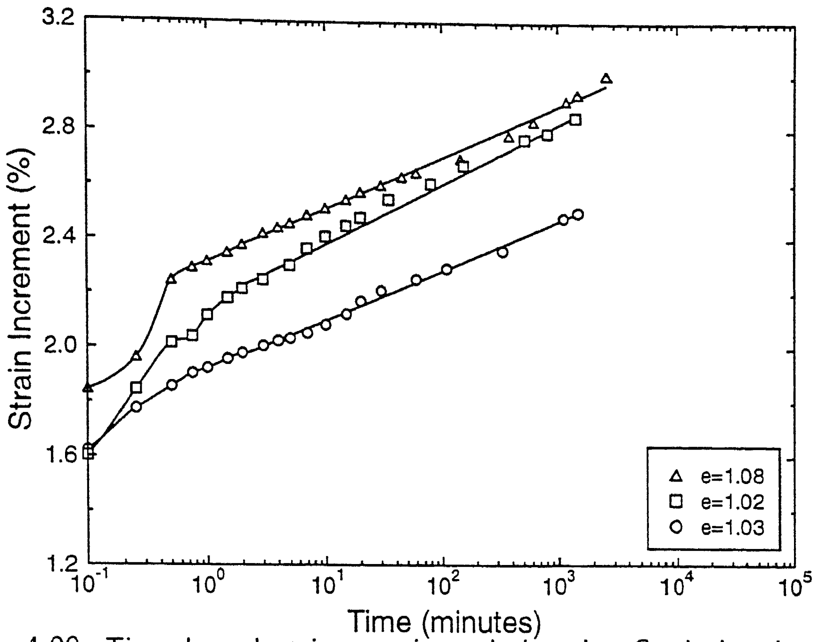

Figure 4.29: Time dependent increase in vertical strain—Crushed rock, stress increment 1100–1600 kPa.

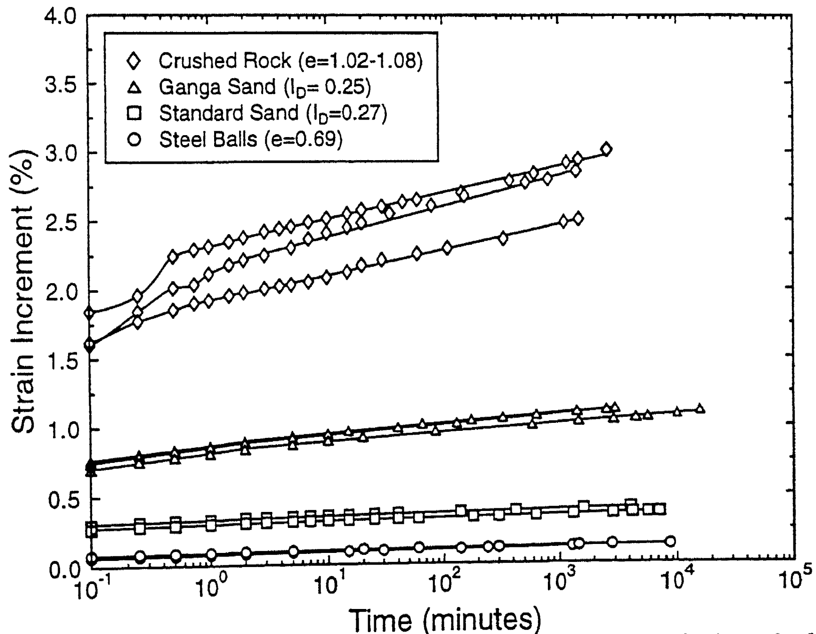

Figure 4.30: Comparison of the time dependent increase in vertical strain for materials with different angularity—stress increment 1100–1600 kPa.

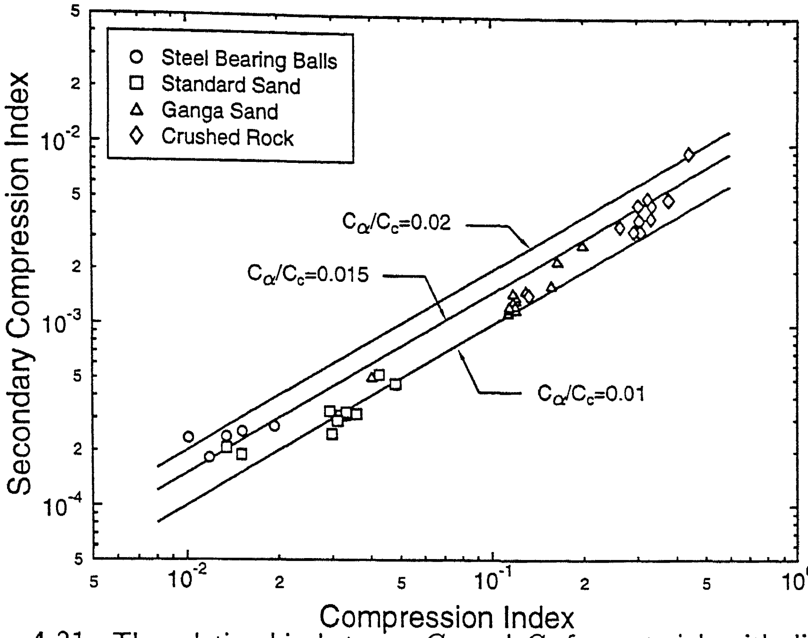

Figure 4.31: The relationship between C_α and C_c for materials with different morphology.

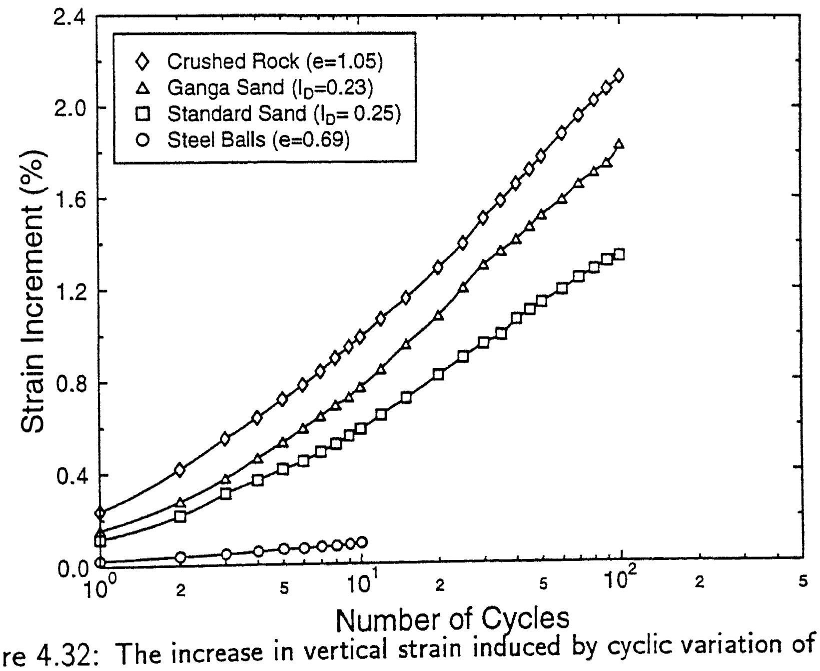

Figure 4.32: The increase in vertical strain induced by cyclic variation of the vertical stress between 400 and 1600 kPa.

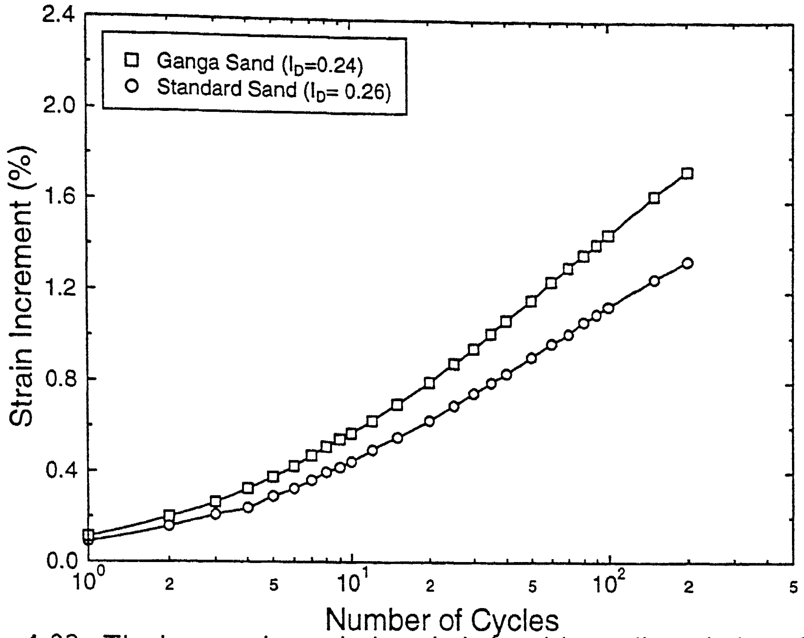

Figure 4.33: The increase in vertical strain induced by cyclic variation of the vertical stress between 100 and 400 kPa.

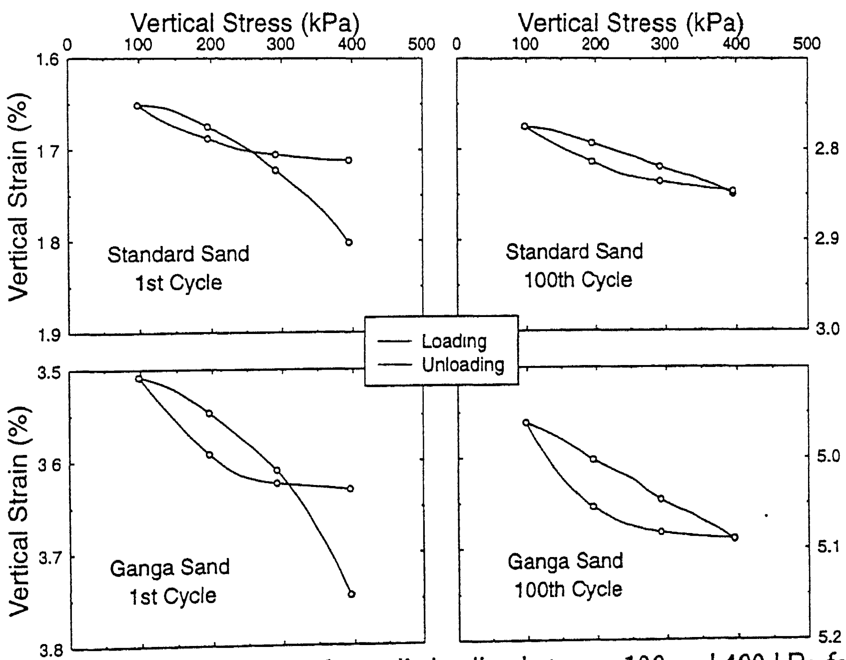

Figure 4.34: Hysteresis loops for cyclic loading between 100 and 400 kPa for Standard and Ganga sands with the same I_D

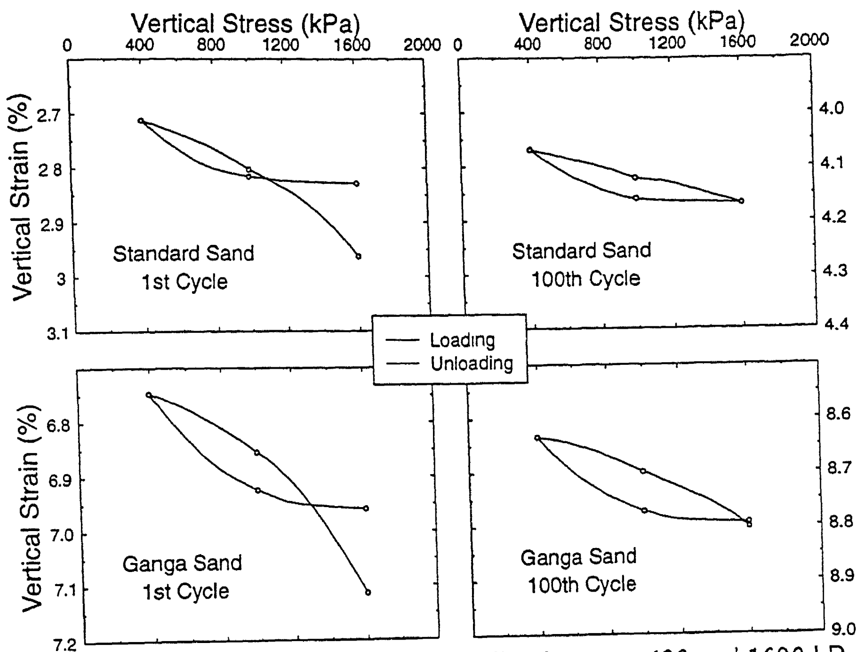

Figure 4.35: Hysteresis loops for cyclic loading between 400 and 1600 kPa for Standard and Ganga sands with the same I_D

4.2 Ageing and Engineering Behaviour

4.2.1 1-D Compression Behaviour

In nature, sands undergo secondary compression following their deposition and this is considered to be an important mechanism of ageing. In the present work, oedometer tests were conducted to study the effects of secondary compression on the stress-strain and strain-time response on subsequent loading. During secondary compression, strains are induced in the sand even though the vertical effective stress remains constant. Hence a major consequence of secondary compression is strain hardening of the soil. There are other mechanisms which can also lead to additional strains without causing a net increase in vertical stress. In this study two such mechanisms namely, overconsolidation and cyclic pre-straining were investigated.

Overconsolidation entails an increase in the vertical stress beyond the current value and then unloading to the original value. In the present context the term cyclic pre-straining is used to denote a cyclic variation of the vertical stress between the current value and a lower value. It is felt that a comparative study of these three phenomena could lead to a better understanding of the nature and consequences of secondary compression. Seismically induced cyclic variation of shear stress may cause strain hardening of natural deposits of sand. Since this also is a time dependent phenomenon occurring under conditions of constant effective over burden stress it may be considered as a part of the ageing process. Oedometer tests involving cyclic pre-straining may be expected to provide some insight into this aspect also. Accordingly a series of oedometer tests were carried out on dry specimens of Ganga sand to study the effects of secondary compression, cyclic pre-straining and overconsolidation on the stress-strain and strain-time response of sands.

Stress-Strain Behaviour

Oedometer tests were carried out on specimens of Ganga sand with $I_D = 0.25$. In tests designated as normal compression tests, the samples were loaded to a vertical stress of 1600 kPa and then the stress was increased in equal steps of

35 kPa, with each increment being added 5 minutes after the application of the previous one. Another series of tests were conducted which were similar to the normal compression tests, except that after loading to 1600 kPa the samples were subjected to secondary compression, cyclic pre-straining or over consolidation and then the vertical stress was increased in small increments as in the case of normal compression tests.

In one test secondary compression was allowed to take place for 11 days under a vertical stress of 1600 kPa. Two tests were carried out to study the effect of cyclic pre-straining wherein after the specimens were loaded to 1600 kPa, the vertical stress was cycled between 1600 kPa for 55 times. Two samples were subjected to overconsolidation by increasing the vertical stress to 1775 kPa and then decreasing it to 1600 kPa. The number of cycles of loading and the overconsolidation ratio were selected so as to result in approximately the same amount of additional strain as caused by 11 days of secondary compression. After the required amount of vertical strain was induced the specimens were loaded beyond 1600 kPa, using the same procedure as in the case of normal compression tests.

The results obtained are presented in Figure 4.36 where the portions of the stress-strain curve beyond 1600 kPa are shown. The ordinate is the vertical strain increment obtained by subtracting the strain corresponding to 5 minutes after the application of 1600 kPa from the total strain. This has been done to eliminate the inevitable scatter resulting from differences between different specimens even though the relative density was approximately same in all cases. It may be seen that secondary compression, cyclic pre-straining and overconsolidation result in increased stiffness, but as stress level increases the compressibility increase. As expected, beyond a certain value of vertical stress the behaviour of all samples become practically identical to that of the normally compressed specimens.

To facilitate a better comparison of the increase in stiffness caused by the three phenomena, the data shown in Figure 4.36 is re-plotted in Figure 4.37, after subtracting the additional strain induced by each mechanism. It may be observed that the response after secondary compression and cyclic

pre-straining is almost identical. But overconsolidated specimens exhibit a stiffer response. For each sequence of loading, a correction for the apparatus compliance was applied using the appropriate calibration curve. The difference in behaviour exhibited by the overconsolidated samples are significantly larger than the plausible error in the calibration. Hence it is believed that the observed difference is unlikely to be due to some error.

4.2.2 Strain-Time Behaviour

The effect of one day of secondary compression at 1600 kPa, on the strain-time response on subsequent loading is illustrated in Figure 4.38. Here the increase in vertical strain due to each vertical stress increment of 35 kPa is plotted against time. The shaded portion represents the range for normally compressed specimens for all increments of 35 kPa between 1600 and 1880 kPa, for which readings were available up to 5 minutes. The data for one normal compression test where for the increment of 1845–1880 kPa strains were recorded for one day is also shown. For the specimens which have experienced one day of secondary compression at 1600 kPa, the strain-time curves for the first few increments are relatively flat. As the stress level increases the curves steepen and approach that of the normally compressed specimens. For the increment of 1845–1880 kPa, the behaviour is not significantly different from that of the normally compressed specimens.

The observed response is in accordance with Mesri's postulate that for a given sand C_α/C_c = constant. When the aged specimens are loaded beyond 1600 kPa, initially they exhibit a stiff response and as the stress level increases further, the behaviour tends to that typical of normally compressed samples. This increase in compressibility is being reflected in the increasing slopes of strain-time curves. This type of tests constitute a simple and elegant way of demonstrating the validity of the concept of C_α/C_c being a constant for a given sand.

The effect of secondary compression, cyclic pre-straining and overconsolidation (with each mechanism resulting in approximately the same amount of additional strain at 1600 kPa) is illustrated in Figures 4.39, 4.40 and 4.41

respectively. Each figure depicts the increase in strain with time up to 5 minutes for each increment of 35 kPa. Figure 4.39 shows the behaviour of the specimen that was allowed to experience 11 days of secondary compression at 1600 kPa and then loaded further in equal steps of 35 kPa in intervals of 5 minutes. As the stress level increases, the strain-time curves continuously steepen and finally approach the range for normally compressed specimens. The results obtained for two tests in which the specimens were subjected to 55 cycles of loading and unloading between 1100 and 1600 kPa and then loaded further, also present a similar picture (Figure 4.40). The response of overconsolidated specimens which were first loaded to 1775 kPa, unloaded to 1600 kPa and then reloaded is shown in Figure 4.41. The broad pattern of behaviour is found to be similar in many respects to the aged and cyclically pre-strained specimens. However, for the first few increments the curves are flatter than the corresponding curves for aged and cyclically pre-strained specimens.

The data presented in Figures 4.39, 4.40 and 4.41 is reproduced in Figure 4.42, where the strain-time behaviour after secondary compression, cyclic pre-straining and overconsolidation are compared for each increment of stress. It may be observed that the effects of secondary compression and cyclic pre-straining on subsequent strain-time response are found to be practically identical at all stages. But until a stress of 1705 kPa, the increase in strain with time is suppressed to a much larger extent in the case of overconsolidated specimens. Thus the similarity between strain-time response after secondary compression and cyclic pre-straining reinforces the observation made earlier that the consequences of these phenomena are practically identical with respect to stress-strain behaviour. On the other hand, the overconsolidated specimens are again found to exhibit a stiffer response.

In this series of tests, after the samples were loaded to 1600 kPa, equal amounts of additional strains were induced by secondary compression, cyclic pre-straining and overconsolidation. On further loading the aged and cyclically pre-strained specimens were found to exhibit practically identical stress-strain and strain-time response, whereas the behaviour of overconsol-

idated samples was significantly different. Thus it is seen that even though the strain induced by the three mechanisms is the same, the consequences of overconsolidation is different from the other two. In the case of aged and cyclically pre-strained specimens yielding begins at a lower stress in comparison to the overconsolidated ones, as indicated by the stress-strain and strain-time curves. Apparently the expansion of the yield locus caused by secondary compression and cyclic pre-straining is of a lower magnitude than that due to overconsolidation.

The observed similarity between one-dimensional stress-strain and strain-time response of aged and cyclically pre-strained specimens suggests that cyclic pre-straining may be employed to simulate the effects of secondary compression in the laboratory. It is important that the stress range in which cyclic loading is carried out, be kept sufficiently small. It was found that when the stresses were cycled over a larger range (400–1600 kPa), the samples exhibited a much softer response. It is clear that the smaller this range, the closer would be the similarity between secondary compression and cyclic pre-straining.

4.2.3 Behaviour in Direct Shear

To explore further the consequences of secondary compression and cyclic pre-straining, direct shear tests were carried out on loose samples of Ganga sand. All specimens had approximately the same relative density (0.24–0.27) and were sheared under a normal stress of 800 kPa. In the first test the sample was sheared 5 minutes after the application of the normal stress (normal compression). Two tests were performed in which the specimen was allowed to age for 2 hours and 11 days after the application of normal stress and then sheared. To study the effect of cyclic pre-straining, a sample was subjected to 50 cycles of loading and unloading of the normal stress between 400 and 800 kPa prior to shearing. Because of some difficulties in the measurement of apparatus compliance, the exact magnitude of vertical strains induced by secondary compression and cyclic pre-straining could not be determined. But it is thought that the strain developed due to 50 cycles of pre-straining was

more than that caused by 11 days of secondary compression.

The results are presented in Figure 4.43 where the shear stress normalized with respect to the normal stress (τ/σ), and the vertical compression are plotted against the horizontal displacement. The maximum value of τ/σ is found to be same in all the tests, indicating that the angle of shearing resistance was not affected by secondary compression or cyclic pre-straining. The behaviour of the normally compressed specimen and the one subjected to 2 hours of secondary compression is practically the same. 11 days of ageing and 50 cycles of pre-straining appear to result in a slight increase in stiffness, although the effect is marginal. The consequences of secondary compression and cyclic pre-straining are more readily noticed in relation to the volume contractions during shear. Here also the specimens which were normally compressed and aged for 2 hours are found to exhibit identical behaviour. But the specimen which was aged for 11 days is seen to be less contractive and the cyclically pre-strained specimen shows the maximum reduction in contractancy.

Daramola (1980) carried out drained triaxial tests on isotropically consolidated specimens of a sand which were aged for 0, 10, 30 and 152 days before shearing. He found the behaviour of the samples aged for 0 and 10 days to be practically identical. However, longer periods of ageing were found to result in significant increase in stiffness and reduction in contractancy even though the increase in the angle of shearing resistance was marginal. The results of the direct shear tests performed in this study appears to be in broad agreement with the findings of Daramola, although 10 days of secondary compression resulted in a noticeable reduction in contractancy. The main consequences of ageing would be a reduction in contractancy and an increase in the deformation modulus. To clearly bring out the increase in stiffness, more accuracy in the experimental procedures and longer periods of ageing are required.

4.2.4 Effects of Ageing, Cyclic Pre-straining, Overconsolidation and Cementation

A considerable amount of information on the effects of cyclic pre-straining (under both triaxial and one-dimensional conditions), overconsolidation and cementation is available in literature. A comparison of the impact of these phenomena on different aspects of the strength and deformation behaviour of sands with that of ageing can provide some insight into the mechanisms of ageing. Hence in the following subsubsections the available information on this topic is summarized.

Critical Stress

It is well-known that overconsolidation results in the expansion of the yield locus of soils. In the oedometer test this is manifested in the form of a critical stress which demarcates the stress range up to which the soil exhibits a stiff response. Strain hardening induced by secondary compression and cyclic pre-straining also leads to the development of a critical stress. For all the three mechanisms of strain hardening, the normal compression curve represents a state boundary. Formation of inter-particle cohesive bonds also effects a critical stress, but in this case it is possible for the stress-strain curve to cross the normal compression curve. Mesri (1987) presents a theoretical relationship for estimating the ratio of the critical stress induced by secondary compression to the vertical effective stress corresponding to the end of primary consolidation. This ratio may be considered analogous to OCR of overconsolidated soils. For clays Mesri (1987) suggests a value of 2 as the maximum possible OCR resulting from secondary compression. One of the factors which governs this ratio is the magnitude of C_α/C_c. Since sands have lower values of C_α/C_c, in this case the maximum possible OCR caused by secondary compression would be less than that for clays.

Maximum Shear Modulus

Resonant column tests carried out on dry specimens of sands by Afifi and Woods (1971), Afifi and Richart (1973) and Fioravante et al. (1994) show that the increase in maximum shear modulus (G_{max}) per log cycle of time is only of the order of about 3% of the value corresponding to a time ranging from 5 to 1000 minutes. This increase can be considered to be due to secondary compression alone. Jamiolkowski and Manassaro (1995) state that, for silica sands the rate of increase of G_{max} per log cycle of time, observed in the laboratory is about 1.1 to 3.5% of the value at the end of primary consolidation whereas for carbonate sands it ranged from 5.3–12%. Jamiolkowski et al. (1988), Berardi et al. (1991) and Fahey and Soliman (1994), on the basis of in-situ measurements on aged natural deposits of sand, suggest that ageing leads to some increase in G_{max}.

Tatsuoka and Shibuya (1992) and Teachavorasinskun et al. (1994) found that G_{max} is not significantly influenced by pre-straining. Afifi and Richart (1973), Tatsuoka and Shibuya (1992), Porovic and Jardine (1994) and Teachavorasinskun et al. (1994) report that the effect of overconsolidation on G_{max} is negligible. Experiments performed on artificially cemented sands, by Acar and El-Tahir (1986) and Chang and Woods (1992) reveal that cementation leads to considerable increase in G_{max} depending on many factors like the type of sand, relative density and the nature and quantity of cementing agent.

Deformation Modulus

Here the deformation modulus (E) refers to the tangent or secant modulus as would be determined in a conventional triaxial test. It corresponds to axial strains of the order of about 10^{-3}–10^{-2}. Results presented by Daramola (1980) indicate that ageing results in significant increase in deformation modulus. On the basis of the data for 152 days he suggested that there would be a 50% increase in E per log cycle of time. On the other hand, Jamiolkowski et al. (1985) state that the limited experimental data available appeared to imply that the influence of secondary compression on the maximum deformation modulus (E_{max}) which corresponds to axial strains less than 10^{-5}, is

comparable to that on G_{max}.

Tatsuoka and Shibuya (1992) and Teachavorasinskun et al. (1994) report that cyclic pre-straining leads to a considerable increase in deformation modulus. But they found that E_{max} is not affected by cyclic pre-straining. The effect of overconsolidation was investigated by Lambrechts and Leonards (1978), Clayton et al. (1985) and Bellotti et al. (1985) by pre-stressing specimens along various paths. They found that pre-stressing increased deformation modulus by as much as an order of magnitude.

Clough et al. (1981) examined the effect of cementation on deformation modulus by conducting tests on undisturbed samples of natural sands with varying degrees of cementation and also on specimens of artificially cemented sands. They found that depending on the nature and degree of cementation. there was a considerable increase in deformation modulus. They also report that the behaviour of naturally and artificially cemented sands is qualitatively similar.

Volume Changes During Shear

The findings of Daramola (1980) and the results of the direct shear tests carried out in the present work indicate that ageing suppresses to some extent the tendency to decrease in volume during shear. However, in the case of locked sands because of their large relative densities and highly developed inter-penetrative fabric there is a phenomenal increase in positive dilatancy with age (Dusseault and Morgenstern 1978, 1979. Barton et al. 1986, Palmer and Barton 1987). Investigations by Teachavorasinskun (1994) and also the results of direct shear tests from the present study reveal that cyclic pre-straining reduces volume contraction during shear. Teachavorasinskun (1994) cites the work of Teachavorasinskun (1989) and Park (1992) to show that overconsolidation causes the behaviour to be less contractive during shear. Clough et al. (1981) report that volume increase during shear occur at a faster rate and at a smaller strain for cemented sands than for uncemented sands of comparable relative density. Thus cementation enhances positive dilatancy.

Drained Angle of Shearing Resistance

Investigations by Daramola (1980) and the results of direct shear tests carried out in the course of the present work indicate that under normal circumstances ageing does not lead to an increase in ϕ'. But in the case of locked sands there is, often a tremendous increase in ϕ'. For the Canadian locked sands, values of the order of 55–65° have been reported (Dusseault and Morgenstern 1978, 1979). But for the British locked sands somewhat lower values were indicated (Palmer and Barton 1987).

Teachavorasinskun (1994) reports that cyclic pre-straining does not influence ϕ'. Lambrechts and Leonards (1978) state that stress history did not influence the drained shear strength of sands. Bellotti et al. (1985) examined the effect of overconsolidation on ϕ' and found the effect to be negligible. The results of the studies on artificially cemented sands conducted by Clough et al. (1981) and Rad and Tumay (1986) show that the effect of cementation on peak and ultimate angles of shearing resistance is not significant.

Penetration Resistance

Durante and Voronkevich (1955), Dudler et al. (1968), Dudler and Iulin (1981) and Mitchell and Solymar (1984) report that the penetration resistance of sand-fills after placement, increases considerably over a period of time ranging from a few months to a few years. Skempton (1986) and Barton et al. (1988) present data showing significant effect of ageing on SPT N values. Troncoso (1994) found penetration resistance to increase with age of deposits in tailing dams. But Baldi et al. (1988) and Berardi et al. (1991) suggest that the effect of ageing on penetration resistance would be less in comparison to the effect on stiffness. Many researchers have also presented evidence for the significant increase in penetration resistance with time after the densification of ground by blasting, heavy tamping etc. (Mitchell and Solymar 1984, Dowding and Hryciw 1986, Schmertmann 1987, Charlie et al. 1992).

Bellotti et al. (1985) examined the effect of cyclic pre-straining along the K_0 path and found that while there was a four fold increase in deformation

modulus, cone tip resistance did not increase significantly. Lambrechts and Leonards (1978) using model cone tests and Clayton et al. (1985) and Bellotti et al. (1985) using chamber tests studied the influence of stress history on penetration resistance. They found that the only consequence of overconsolidation, which leads to an increase in penetration resistance is the increase in horizontal stress. When the effect of horizontal stress is accounted for, the behaviour of normally consolidated and overconsolidated sands are identical with respect to penetration resistance. Thus pure strain hardening with out any associated increase in lateral stress either by cyclic pre-straining or overconsolidation does not have a significant effect on penetration resistance.

Investigations by Rad and Tumay (1986) on artificially cemented sands show that cementation has a pronounced influence on penetration resistance. They found that as cement content increases, both cone tip resistance and sleeve friction increase while friction ratio decreases. They also state that several researchers in the past have noticed similar effects of cementation in naturally deposited sands. Puppala et al. (1995) report the results of studies on artificially sands in a calibration chamber. They also found both cone tip resistance and sleeve friction to increase due to cementation, but did not observe any significant change in friction ratio in the vertical stress range of 100–300 kPa. The effect of cementation on penetration resistance was found to be increasingly suppressed as confining stress increased.

Resistance to Liquefaction

Mulilis et al. (1977) found that the cyclic stress ratio required to cause 100% pore pressure response in 10 cycles of loading, increased with the period of time for which the specimen was subjected to the confining stress. Similar finding by Banerjee et al. (1979) for gravels is cited by Tatsuoka et al. (1988). Tatsuoka et al. (1988) studied the effect of consolidation age and found that, the number of cycles of loading to produce a double amplitude axial strain of 10% for a given cyclic stress ratio, increased with age of the samples. Cassagrande (1975), Youd and Perkins (1978), Seed (1979) and Troncoso (1994) have also suggested that the age of the deposit enhances resistance to

liquefaction.

Teachavorasinskun et al. (1994) report that, the resistance to liquefaction as represented by the number of cycles of loading required to cause a double amplitude axial strain of 5%, increases due to cyclic pre-straining. Tatsuoka et al. (1992) cite similar results obtained by Kenkyo et al. (1991). Many researchers have shown that the resistance to liquefaction of sands is considerably enhanced by overconsolidation (Seed and Peacock 1971, Lee and Focht 1975, Ishihara and Okada 1978, Ishihara and Takatsu 1979 and Tatsuoka et al. 1988). The work of Ishihara and Takatsu (1979) indicate that the beneficial effects overconsolidation arise both from strain hardening and increase in K_0.

Clough et al. (1989) reviews the work of several researchers who have suggested that as the degree of cementation increases, resistance to liquefaction also increases. They also investigated the behaviour of artificially cemented specimens of sand having the same relative density. It was found that even a small amount of cementation leads to a considerable increase in the number of cycles of loading required to cause initial liquefaction for a given cyclic stress ratio.

4.2.5 Mechanisms of Ageing Under Geo-static Conditions

Subsequent to deposition, sands experience ageing which leads to significant improvements in behaviour. Many researchers have ascribed these changes to different phenomena, the most important being secondary compression, cementation and pressure solution and crystal over-growth. Since seismically induced cyclic pre-straining is a time dependent phenomena occurring under conditions of constant effective over burden stress, this may also be considered as an important mechanism of ageing. In order to understand the nature and consequences of ageing at a fundamental level and to develop mechanistic models to predict the response of aged sands, it is necessary to elucidate the predominant mechanisms of ageing. While this problem can be

approached in several ways, in this study an attempt was made to analyse the available information on the known effects of some of these phenomena on different aspects of sand behaviour, so as to gain some insight into the mechanisms of ageing.

Secondary Compression and Cyclic Pre-straining

The strains induced during secondary compression and cyclic pre-straining result in strain hardening and cause the yield locus to expand. There is some evidence to indicate that the lateral stresses may also increase. Therefore the effects are in many respects qualitatively similar to that of overconsolidation. On the basis of the review of the effects of cyclic pre-straining and overconsolidation on the engineering behaviour of sands, the expected consequences of secondary compression and cyclic pre-straining may be summarized as follows. Secondary compression and cyclic pre-straining would result in considerable reduction in volume contractions during shear and significant increase in deformation modulus. But the effect on the maximum shear and deformation modulus and the drained angle of shearing resistance would not be significant. The resistance to liquefaction as represented by the number of cycles of loading required to cause 100% pore pressure response or a particular value of axial strain, would also be affected noticeably. However, the impact of these phenomena on the steady state shear strength is yet to be ascertained. Any increase in penetration resistance can occur only as a result of increase in horizontal stresses.

The observed pattern of behaviour may be explained in terms of the consequences of strain hardening. What strain hardening does is to make the response of sands to be less contractive. Since G_{max} is measured at extremely low strain levels at which gross inter-particle slips have not yet been initiated, it is not affected by strain hardening. However, strain hardening has a tremendous impact on the strain level dependency of the modulus and this is manifested as an increase in modulus at higher strain levels. When loaded beyond the yield locus the response is similar to that of an ordinary sand and hence there is no increase in drained shear strength. However,

because of the reduction in contractancy less pore pressures are induced during undrained shearing and this is the reason for the increase in resistance to liquefaction.

To a large extent the effect of ageing on most aspects of behaviour appears to be similar to that of secondary compression and cyclic pre-straining, indicating that these phenomena could be potential mechanisms of ageing. The only discrepancy seems to be the relatively large increase in penetration resistance reported by several researchers. Mitchell and Solymar (1984) report that cone penetration resistance measured 50–80 days after placement of a hydraulic sand-fill, was about 100% more than the value recorded 4–10 days after placement of the fill. Durante and Voronkevich (1955) found that the penetration resistance of a natural sand deposit in the undisturbed state to be 70–100% more than the value observed when the same sand was freshly deposited and compacted to the same density. Dudler and Iulin (1981) found the penetration resistance of a sand-fill to increase by a factor of 2.4 over a period of 80 months.

To examine whether secondary compression can lead to such large increase in penetration resistance, the following approach is suggested for obtaining an approximate estimate of the possible increase in cone tip resistance. Bellotti et al. (1985) present the following empirical equation developed by Schmertmann which relates the cone tip resistance of overconsolidated sands ($q_{c,oc}$) to that of normally consolidated sands ($q_{c,nc}$).

$$\frac{q_{c,oc}}{q_{c,nc}} = 1 + \chi(\frac{K_{0,oc}}{K_{0,nc}} - 1) \tag{4.20}$$

where $K_{0,oc}$ and $K_{0,nc}$ are the coefficients of earth pressure at rest for overconsolidated and normally consolidated sand respectively. χ is an empirical factor for which Schmertmann recommended a value of 0.75. Bellotti et al. (1985) found that for Ticino sand with an OCR of 2, χ had a range of 0.4–1.0 with an average value of 0.65.

Because both in the case of secondary compression and overconsolidation an increase in penetration resistance can result only from an increase in K_0, the above equation may be used to estimate the effect of secondary

compression on q_c. Following Mesri and Hayat (1993), the value of K_0 at any instant of time t may be expressed in terms of the value corresponding to a reference time t_R.

$$\frac{K_{0,t}}{K_{0,t_R}} = (\frac{t}{t_R})^{C_\alpha/C_c} \tag{4.21}$$

With $t = 10$ years, $t_R = 1$ day and $C_\alpha\ C_c = 0.02$ increase in K_0 in 10 years is computed to be 18%. Even if the maximum reported value of 1 is assumed for χ, increase in q_c over a period of 10 years works out to be only 18%.

Hence it is clear that an increase in penetration resistance of the order of 100% or more during time periods less than 10 years, cannot be ascribed to the effects of secondary compression alone. It appears that some other mechanism is involved which causes the major increase in resistance.

Cementation

Cementation leads to development of cohesive bonds at inter-particle contacts and filling of void spaces depending on the degree of cementation. Mitchell and Solymar (1984) and Denisov and Reltov (1961) suggest that cohesive bonds may form between sand grains because of the gradual joining of the silicic acid gel films generated at the surface of the grains as a result of chemical interaction between water and silica, and also due to precipitation of silica or other materials from solution or suspension at the particle contacts. These researchers propose cementation as the basic mechanism of ageing of sands.

The review of the effect of cementation presented earlier indicates considerable increase in maximum shear modulus, deformation modulus, drained shear strength, penetration resistance and liquefaction resistance. Ageing leads to significant increase in deformation modulus, liquefaction resistance and often in penetration resistance. But tests conducted by Daramola (1980) on saturated samples of a sand which was aged for a period of 150 days did not show any increase in drained shear strength, indicating that in this case cementation did not develop. The effect of ageing on G_{max} appears to be quite small in comparison to that on deformation modulus at higher strain

levels. Jamiolkowski and Manassaro (1995) report that the increase in G_{max} per log cycle of time is only of the order of 3%, whereas Daramola (1980) predicts that E would increase by 50% per log cycle of time. If cementation is the predominant mechanism, then there should be a significant increase in maximum shear modulus also. But as most of the laboratory studies on the time effects on shear modulus were conducted on dry specimens, it could be argued that cementation bonds could not form in such cases. A careful examination of the data from in-situ tests on natural sand deposits may provide further insight into this aspect.

The effects of cementation and ageing appears to be similar in many respects. Several researchers have presented arguments both in favour and against cementation as a mechanism of ageing and these were reviewed in Chapter 2. To conclusively prove that cementation can occur in relatively short time spans, further studies in the field and laboratory are required.

Pressure Solution and Crystal Over-Growth

The effects of ageing are developed to the highest degree in the case of locked sands which are several millions years old. These materials are characterized by very large relative densities and a highly developed inter-penetrative fabric and exhibit very high strength and stiffness. The primary mechanism by which ordinary sands are transformed into locked sands is believed to be the process of pressure solution acting through the long geologic history of these deposits, even though in some cases crystal over-growth may also be important (Dusseault and Morgenstern 1978, 1979, Barton et al. 1986, Palmer and Barton 1987 and Barton and Palmer 1989).

Palmer and Barton (1987) present quantitative evidence for the change in the type of contact from predominantly tangential and short-straight to long-straight, concavo-convex and sutured, with age for British sands. On the basis of the data presented by Palmer and Barton (1987), Yudhbir and Rahim (1993) argued that the sum of long-straight, concavo-convex and sutured contacts could be taken as a measure of the average percentage of gain surfaces in contact for the aged sands. They defined a grain contact index

(I_c) which is obtained by subtracting the sum of the percentages of tangential and short-straight contacts from 100. For the British aged sands, the ratio of shear stress to normal stress (τ/σ) corresponding to σ = 200 kPa and the unconfined compressive strength are found to be strongly correlated to I_c. This lends credence to the point of view that the high strength and stiffness of locked sands is to a large extent due to the highly developed micro-interlocking of the grains.

The effect of the degree of pressure solution on the shearing response of locked sands of the same age and having the same mineralogy and porosity, is illustrated by the behaviour of Swan River sand (Dusseault and Morgenstern) and Woburn sand (Palmer and Barton 1987). Both are Lower Cretaceous, quartzose sands with an in-situ porosity of 0.345. While for Swan River sand τ/σ corresponding to σ = 200 kPa was 2.11, a considerably lower value of 1.39 was observed for Woburn sand. This is because of the occurrence of more intense pressure solution in the case of Swan River sand.

4.2.6 Post Densification Penetration Resistance

Densification of cohesionless deposits by methods like blasting, heavy tamping and vibro-compaction etc. has often been found to result in considerable reduction in penetration resistance immediately after the treatment, even though significant densification had occurred. But the penetration resistance increases with time and often exceeds the initial value by a large margin. Prediction of the increase in penetration resistance with time is essential for the evaluation of the efficacy of the densification programme and also for design of foundations on densified ground. It is clear that in order to develop a mechanistic model, the relevant mechanism needs to be identified. The first attempt in this direction was made by Mesri et al. (1990) who assumed secondary compression to be the dominant mechanism. They suggest that the increase in penetration resistance with time is due to the enhanced macro-interlocking of grains and micro-interlocking of grain surface roughnesses and the increase in horizontal effective stress associated with secondary compression.

Mesri et al (1990) assume that

$$q_c \propto (M\sigma'_h)^{0.5} \tag{4.22}$$

To estimate the increase in constrained modulus (M) during secondary compression the following equation was proposed.

$$\frac{M}{M_p} = \frac{1}{C_r/C_c}(\frac{t}{t_p})^{C_D C_\alpha/C_c} \tag{4.23}$$

where M_p is the constrained modulus corresponding to end of primary consolidation t_p, M is the value at time t and C_D is an empirical constant which accounts for the effect of any disturbance to the structure of the sand. The increase in horizontal stress was estimated using

$$\frac{\sigma'_h}{\sigma'_{v0}} = K_{0p}(\frac{t}{t_p})^{C_D C_\alpha/C_c} \tag{4.24}$$

where K_{0p} is the value of K_0 at time t_p. Substituting Equation 4.23 and Equation 4.24 into Equation 4.22, Mesri et al. (1990) derived the following equation.

$$\frac{q_c}{(q_c)_R} = (\frac{t}{t_R})^{C_D C_\alpha/C_C} \tag{4.25}$$

where $(q_c)_R$ is the penetration resistance at a reference time t_R.

The basic assumption invoked by Mesri et al. (1990) (Equation 4.22) appears to contradict the results of the investigations on the effect of overconsolidation on penetration resistance (Lambrechts and Leonards 1978, Clayton et al. 1985, Bellotti et al. 1985). These researchers found that the only outcome of overconsolidation that led to an increase in penetration resistance was the increase in horizontal stress. Pure strain hardening without increase in horizontal stress resulted in a phenomenal increase in modulus, but did not have any significant influence on q_c. As the consequences of secondary compression and overconsolidation may be considered to be qualitatively similar, Equation 4.22 is likely to result in considerable overestimation of the effect of secondary compression on q_c.

Mesri et al. (1990) evaluated C_D using Equation 4.23 by conducting oedometer tests. The effect of ground densification was simulated by densifying

the specimen by tapping the sides of the container. Then the sample was allowed to undergo secondary compression for 24 hours and loaded further in small increments. The value of M_p was determined from a normal compression test where each increment was added 5 minutes after the application of the previous one. Substituting the values of M and M_p into Equation 4.23, the value of C_D was computed and this was presumed to be applicable to in-situ ground conditions after densification. It is clear that C_D determined in such a manner reflects the immediate increase in constrained modulus which is induced by the densification, in addition to the time dependent increase resulting from secondary compression. A higher value of C_D would be indicated which is partly due to the effect of densification which occurs immediately. By using this value of C_D the effect of secondary compression on the rate of increase of penetration resistance with time may be over estimated.

Mesri et al. (1990) used Equation 4.25 along with the values of C_D determined as described above, to predict the increase in q_c with time observed to occur after densification of the ground. They found that in some instances the predictions agreed with field data, but often the rates of increase observed in the field were much higher. They computed C_D by fitting Equation 4.25 to the field data and found that in general higher values of C_D were observed in the field. Thus in spite of important limitations which would lead to an overestimation of the effects of secondary compression, the increase in q_c predicted by this model are considerably lower than the observed rates in the field. Therefore mechanisms other than secondary compression may also contribute substantially to increase in q_c. It may also be noted that the decrease in penetration resistance immediately following blasting cannot be explained, if secondary compression is the only mechanism involved.

Mitchell and Solymar (1984) suggested that cementation may be the primary cause for the increase in penetration resistance following disturbance to the ground. This hypothesis can explain both the immediate decrease in penetration resistance and the subsequent increase with time. But it remains to be shown that cementation bonds indeed can form in the rel-

atively short time periods involved. For an uncemented sand q_c is primarily a function of the horizontal effective stress and the relative density. Densification of sands occur immediately up on ground treatment and with time there is no real gain in relative density. If cementation is not a significant mechanism, then the only factor which can lead to major changes in q_c is horizontal stress. Solymar (1984) and Schmertmann (1987) suggest that blast densification might produce a macro-arching effect resulting in a temporary decrease in horizontal stress in the region surrounding the points at which detonation occurred. This can explain the immediate reduction in q_c. With time as the horizontal stresses recover, the penetration resistance increases and finally attains a value consistent with the increased relative density. A careful examination of the field data to see whether the final values of the penetration resistance are consistent with the increased relative density, may help to clarify some of these issues.

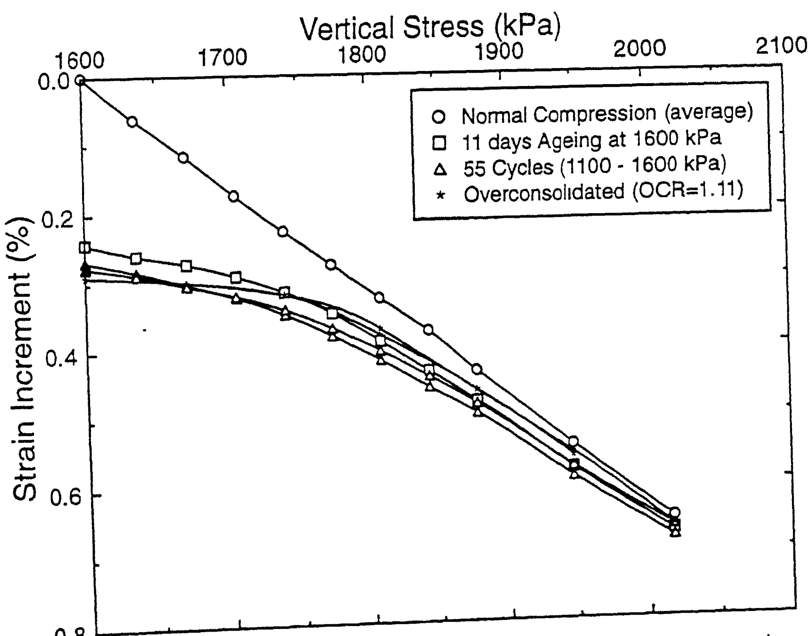

Figure 4.36: Effect of secondary compression, cyclic pre-straining and overconsolidation on the stress-strain response of Ganga sand during subsequent loading.

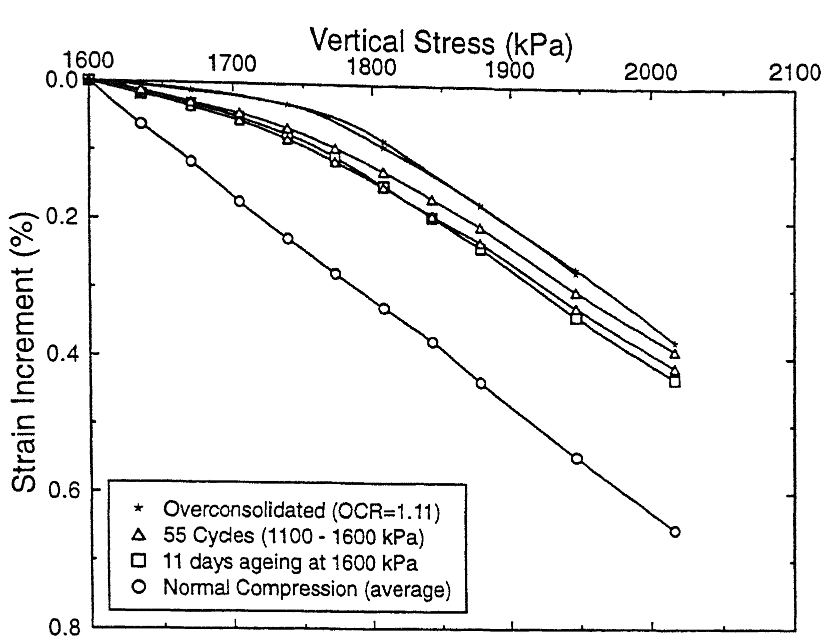

Figure 4.37: Comparison of the stress-strain curves of aged, cyclically pre-strained and overconsolidated specimens of Ganga sand.

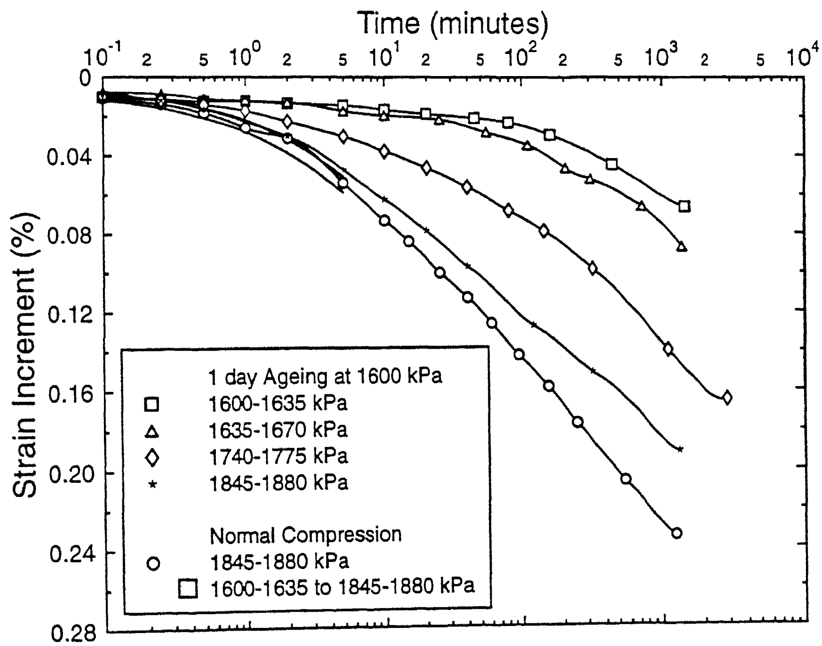

Figure 4.38: Strain-time response of normally compressed and aged specimens of Ganga sand for different stress increments.

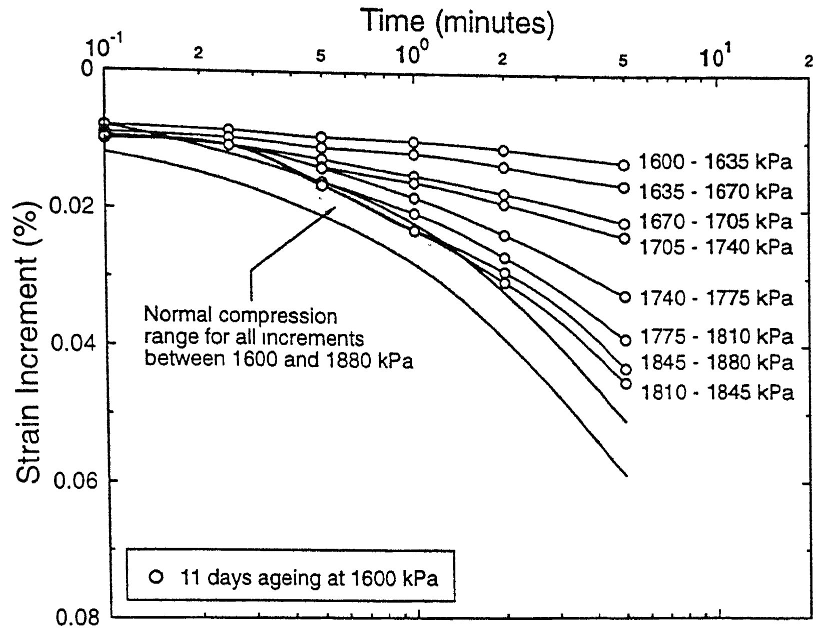

Figure 4.39: Strain-time response of aged specimens Ganga sand for different stress increments.

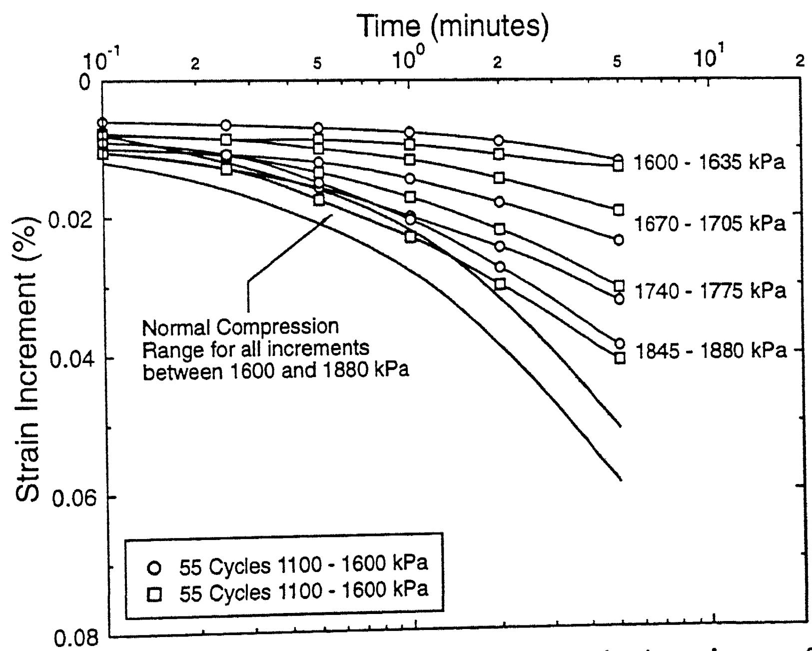

Figure 4.40: Strain-time response of cyclically pre-strained specimens of Ganga sand for different stress increments.

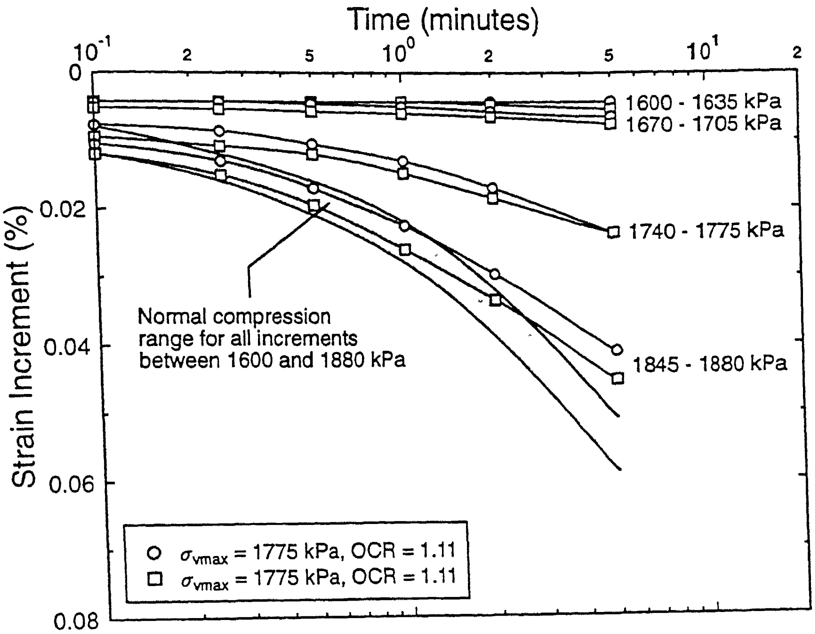

Figure 4.41: Strain-time response of overconsolidated specimens of Ganga sand for different stress increments.

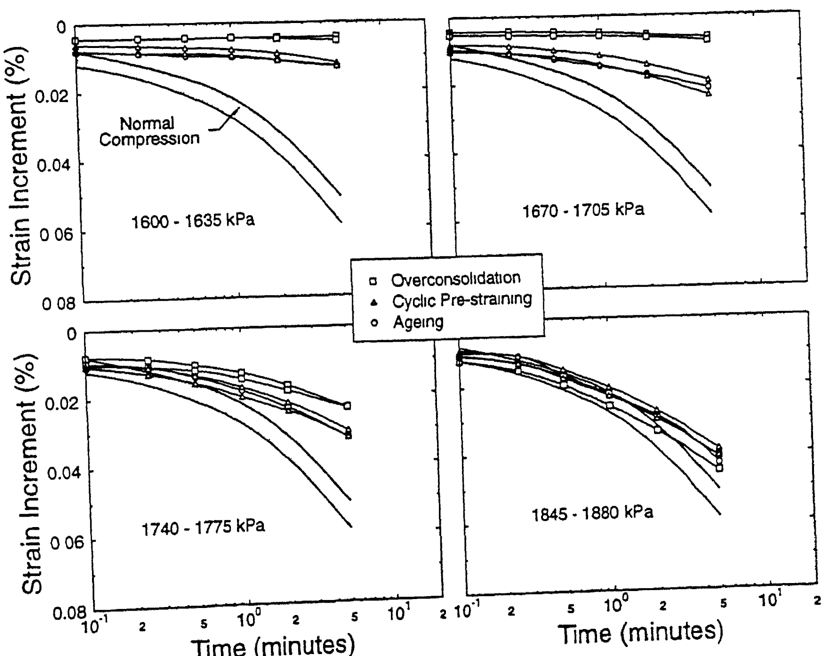

Figure 4.42: Comparison of the effects of secondary compression, cyclic pre-straining and overconsolidation on the strain-time response of Ganga sand for different stress increments.

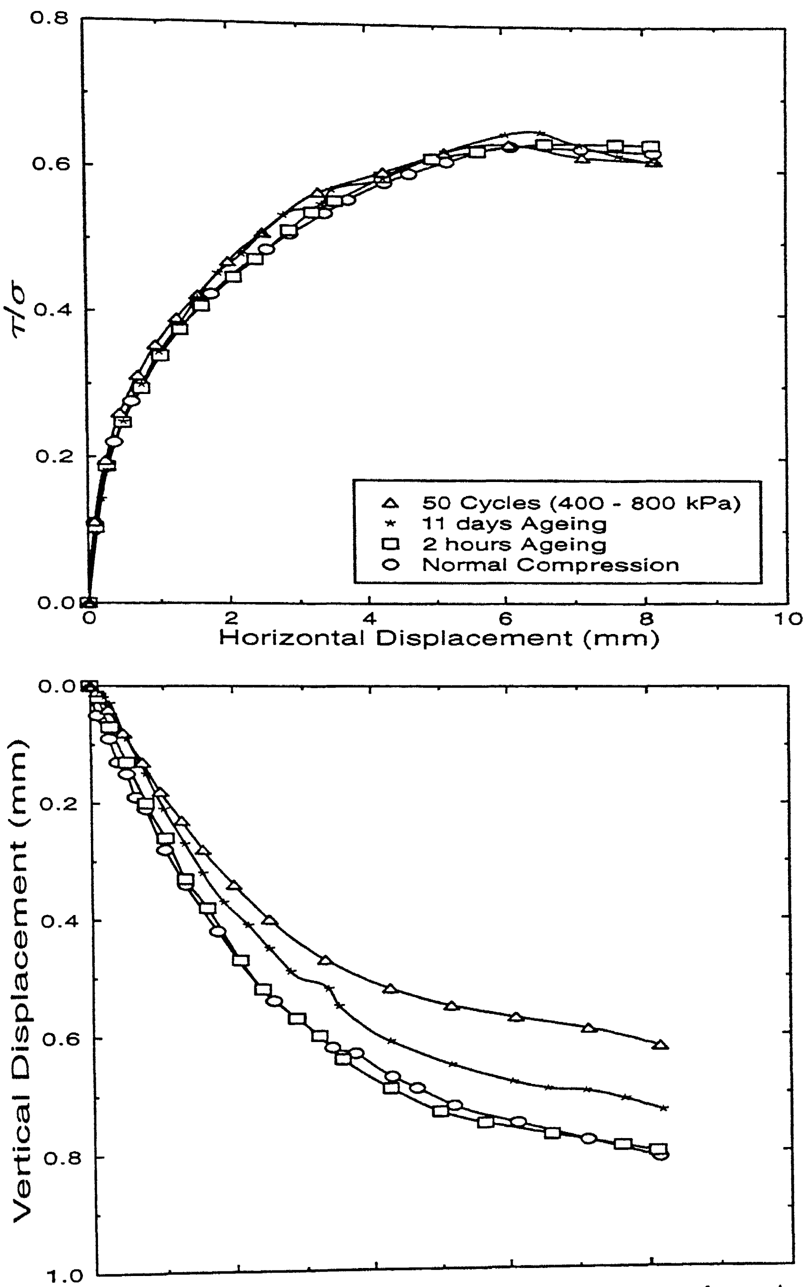

Figure 4.43: Results of direct shear tests on normally compressed, aged and cyclically pre-strained specimens of Ganga sand having the same initial relative density.

Chapter 5

Conclusions

In the present work, an attempt was made to investigate the effect of morphology and ageing on some aspects of the engineering behaviour of sands. Experiments were conducted on materials with a wide range of particle shapes to study the influence of morphology. Experiments have also been carried out to examine the outcome of secondary compression and cyclic pre-straining on the behaviour of sands in one-dimensional compression and direct shear. In addition, some of the available information on these topics has been critically evaluated and interpreted. In this chapter the significant conclusions of this study are presented. Some suggestions for further research in this area are also outlined.

The important conclusions related to morphology and engineering behaviour of sands are as follows.

- The order of magnitude of the grain angularity of a sand can be estimated from information on e_{max}, C_u and D_{50}.

- Differences in behaviour caused by variations in the method of specimen preparation may be more pronounced in the case of angular sands in comparison to rounded sands.

- A fall cone test for the determination of e_{max} has been proposed. This procedure gives values which are practically same as those determined

by the conventional method of pouring. A one-point variant of this method also leads to identical results.

- For most poorly graded, non-micaceous sands there is a strong correlation between e_{max} and e_{min}. This linear relationship may be used to obtain a reasonably accurate estimate of e_{min}.

- Differences in particle angularity can result in changes in ϕ_{cv} of the order of 6°. The higher values of ϕ_{cv} in the case of angular sands are likely to be due to a larger contribution from particle rearrangement, and this appears to be closely related to the more non-uniform distribution of local void ratio.

- For a given magnitude of the confining stress, $e \tan \phi'$ for a sand is a constant. The value of this constant decreases as roundness increases. Based on this concept, a relationship for estimating the angle of shearing resistance has been proposed. This relationship may be used to demonstrate the influence of morphology on ϕ'.

- An analysis of the fall cone test for sands has been presented and this has been used to compute values of N_γ from the results fall cone tests. In the case of a wellrounded sand the relationship between experimentally determined values of N_γ and estimated angles of shearing resistance is in close agreement with a theoretical relationship proposed by Meyerhof (1961). But for an angular sand the agreement is poor.

- The range of initial states corresponding to different types of undrained shear response of saturated sands, is to a large extent controlled by the particle shape. Morphology also has a significant influence on the relationships between some of the parameters characterizing the undrained shear behaviour.

- Roundness has a significant influence on modulus number m_0. It has also some effect on m_2, but m_1 appears to be independent of roundness.

- Morphology affects the nature of the strain-time relationships of sands in one-dimensional compression. However, the effect of particle shape on the value C_α/C_c appears to small.

- Grain angularity leads to significant differences in the magnitude of strains induced and the nature of the hysteresis loops developed during one-dimensional cyclic loading of sands.

The significant conclusions related to ageing and the engineering behaviour of sands are the following.

- The effects of equal amounts of strain induced by cyclic pre-straining within a sufficiently small stress range and secondary compression, on the one-dimensional stress-strain and strain-time response of sands are practically identical. But the outcome of overconsolidation is significantly different.

- Secondary compression and cyclic pre-straining causes the behaviour of sands during shear to be less contractive.

- Cyclic pre-straining using a sufficiently narrow stress range may be employed to simulate the effects of secondary compression in the laboratory because of the observed similarity between their consequences.

- The large time dependent increase in penetration resistance reported to occur in sand-fills after placement cannot be attributed to the effects of secondary compression alone.

- The available evidence does not appear to justify the hypothesis that secondary compression is the predominant mechanism which causes the large time-dependent increase in post densification penetration resistance.

Scope for Future Work

In order to gain a deeper understanding of the effects of morphology and ageing and to develop more sophisticated methods for the prediction of response, further research on these subjects is necessary. In this context, investigations on the following aspects may be of interest.

- Verification of the applicability of the fall cone test for e_{max} for a wide range of sands.
- Validation of the proposed method for predicting the angle of shearing resistance of sands, using data for different sands.
- A systematic study of the influence of morphology and confining stress on the dilatancy and deformation characteristics of sands under drained conditions.
- A systematic comparative study of the effects of secondary compression, cyclic pre-straining and overconsolidation on the strength and deformation characteristics of sands using more accurate procedures and measurements and longer durations of ageing.
- Investigations to gain further insight into the mechanisms of ageing. Some possible approaches are indicated below.
  - Carrying out IDS tests as suggested by Schmertmann (1991).
  - Examination of the changes in fabric occurring with time, by preparing thin sections.
  - Comparison of the degree of ageing effects in dense and loose sands and in saturated and dry sands.
  - Comparison of the effects of ageing on the stiffness at different strain levels.
  - A more careful analysis of the field data on post-densification penetration resistance.

References

Acar, Y.B., and El-Tahir, E.A. (1986) Low strain dynamic properties of artificially cemented sand, *Journal of Geotechnical Engineering*, ASCE, **112**(11), 1001–1015.

Afifi, S.S., and Richart, F.E. (1973) Stress-history effects on shear modulus of soils, *Soils and Foundations*, **13**(1), 77–95.

Afifi, S.S., and Woods, R.D. (1971) Long-term pressure effects on shear modulus of soils, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **97**(10), 1445–1460.

Alyanak, I. (1961) Vibration of sands with special reference to the minimum porosity test for sands, *Proceedings of The Midland Soil Mechanics and Foundation Engineering Society*, **4**, 37–72.

Arthur, J.R.F., Chua, K.S., and Dunstan, T. (1977) Induced anisotropy in a sand, *Geotechnique*, **27**(1), 13–30.

Arthur, J.R.F., Chua, K.S., Dunstan, T., and del Rodriguez, C.J.I. (1980) Principal stress rotation: a missing parameter, *Journal of the Geotechnical Engineering Division*, ASCE, **106**(4), 419–433.

Arthur, J.R.F., and Menzies, B.K. (1972) Inherent anisotropy in a sand, *Geotechnique*, **22**(1), 115–129.

Baldi, G., Bellotti, R., Ghionna, N., and Jamiolkowski, M. (1988) Stiffness of sands from CPT, SPT and DMT—a critical review, *Penetration Testing in the U.K.*, Thomas Telford, London, 299–305.

Baldi, G., Bellotti,R., Ghionna,V.N., Jamiolkowski, M., and Lo Presti, D.C.F. (1989) Modulus of sands from CPT's and DMT's, *Proceedings of 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro, **1**, 165–170.

Barton, M.E., Cooper, M.R., and Palmer, S.N. (1988) Diagenetic alteration and micro-structural characteristics of sands: neglected factors in the in-

Prentice-Hall Inc. Eaglewood Cliffs, New Jersey.

Bolton, M.D. (1986) The strength and dilatancy of sands, *Geotechnique*, **36**(1), 65–78.

Brewer, R. (1964) *Fabric and Mineral Analysis of Soils*, Wiley, New York.

Burland, J.B., and Burbidge, M.C. (1985) Settlements of foundations on sand and gravel, *Proceedings of the Institution of Engineers*, **78**(1), 1325–1381.

Cassagrande, A. (1975) Liquefaction and cyclic deformation of sands— a critical review, Harvard Soil Mechanics Series No. 88.

Castro, G. (1969) *Liquefaction of Sands*, Harvard Soil Mechanics Series No. 81.

Castro, G. (1975) Liquefaction and cyclic mobility of saturated sands, *Journal of the Geotechnical Engineering Division*, ASCE, **101**(6), 551–569.

Castro, G., Enos, J.L., Poulos, S.J., and France, J.W. (1982) *Liquefaction Induced by Cyclic Loading*, Report to National Science Foundation, Washington DC, No. NSF/CEE-82018.

Castro, G., and Poulos, S.J. (1977) Factors affecting liquefaction and cyclic mobility, *Journal of the Geotechnical Engineering Division*, ASCE, **103**(6), 501–516.

Chang, T., and Woods, R.D. (1992) Effect of particle contact bond on shear modulus, *Journal of Geotechnical Engineering*, ASCE, **118**(8), 1216–1233.

Charlie, W.A., Rwebyogo, M.F.J., and Doehring, D.O. (1992) Time-dependent cone penetration resistance due to blasting, *Journal of Geotechnical Enginεering*, ASCE, **118**(8), 1200–1215.

Chen, Y.C., Ishibashi, I., and Jenkins, J.T. (1988) Dynamic shear modulus and fabric: Part I, Depositional and induced anisotropy, *Geotechnique*, **38**(1), 25–32.

Clayton, C.R.I., Hababa, M.B., and Simons, N.E. (1985) Dynamic penetration resistance and the prediction of the compressibility of a fine-grained sand—a laboratory study, *Geotechnique*, **35**(1), 19–31.

Clough, G.W., Iwabuchi, J., Rad, N.S., and Kuppuswami, T. (1989) Influence of cementation on liquefaction of sands, *Journal of Geotechnical Engineering*, ASCE, **115**(8), 1102–1117.

Clough, W.C., Sitar, N., Bachus, R.C., and Rad, N.S. (1981) Cemented sands under static loading, *Journal of the Geotechnical Engineering Division*, ASCE, **107**(6), 799–817.

Conforth, D.H. (1973) Prediction of drained shear strength of sands from

relative density measurements, *Evaluation of Relative Density and its Role in Geotechnical Engineering Projects Involving Cohesionless Soils*, ASTM STP 523, 281–303.

Conforth, J.H. (1974) One-dimensional consolidation curves of a medium sand, *Geotechnique*, **24**(4), 678–683.

Daramola, O. (1980) Effect of consolidation age on stiffness of sand, *Geotechnique*, **30**(2), 213–216.

De Beer, E.E. (1963) The scale effect in the transposition of the results of deep sounding tests on the ultimate bearing capacity of piles and caisson foundations, *Geotechnique*, **13**(1), 39–75.

De Jaeger, J. (1991) *Influence de la morphologie des sables sur leur comportement mecanique*, Ph.D Thesis, Universite Catholique de Louvain.

De Jaeger, J. (1994) Influence of grain size and shape on the dry sand behaviour, *Proceedings of 13th International Conference on Soil Mechanics and Foundation Engineering*, New Delhi, India, **1**, 13–16.

Denisov, N.Y., and Reltov, B.F. (1961) The influence of certain processes on the strength of soils. *Proceedings of 5th International Conference on Soil Mechanics and Foundation Engineering*, **1**, 75–78.

Dickin, E.A. (1973) Influence of grain shape and size upon the limiting porosities of sands, *Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils*, ASTM STP 523, 113–120.

Dowding, C.H., and Hryciw, R.D. (1986) A laboratory study of blast densification of sand, *Journal of Geotechnical Engineering*, ASCE, **112**(2), 187–199.

Dusseault, M.B., and Morgenstern, N.R. (1978) Shear strength of Athabasca oil sands, *Canadian Geotechnical Journal*, **15**, 216–238.

Dusseault, M.B., and Morgenstern, N.R. (1979) Locked sands, *Quarterly Journal of Engineering Geology*, **12**, 117–131.

Edil, T.B., and Luh, G-F. (1978) Dynamic modulus and damping relationships for sands, *Proceedings of ASCE Specialty Conference on Earthquake Engineering and Soil Dynamics*, Pasadena, California, **1**, 394–409.

El-Sohby, M.A., and Andrawes, K.Z. (1972) Deformation characteristics of granular materials under hydrostatic compression, *Canadian Geotechnical Journal*, **9**, 338–350.

Fahey, M., and Soliman, A.A. (1994) Measuring the small-strain behaviour of sand in-situ, *Pre-failure Deformation of Geomaterials*, Shibuya, Mitachi and Miura (eds), **1**, 217–222.

Fioravante, V., Capoferri, R., Hameury, O., and Jamiolkowski, M. (1994) De-

formational characteristics of uncemented carbonate Quiou sand, *Pre-failure Deformation of Geomaterials*, Shibuya, Mitachi and Miura (eds), **1**, 55–61.

Frederick, M.R. (1961) Notes on the shape of particles and its influence on the properties of sands, *Proceedings of The Midland Soil Mechanics and Foundation Engineering Society*, **4**, 137–161.

Giroud, J.-P. (1971) Discussion of "Effect of particle characteristics on soil strength by R.M. Koerner", *Journal of Soil Mechanics and Foundations Division*, ASCE, **97**(4), 693–694.

Griffiths, J.C. (1967) *Scientific Method in Analysis of Sediments*, McGraw-Hill Book Company.

Grivas, D.A., and Harr, M.E. (1980) Particle contacts in discrete materials, *Journal of the Geotechnical Engineering Division*, ASCE, **106**(5), 559–564.

Hansbo, S. (1957) A new approach to the determination of the shear strength of clay by the fall-cone test, *Royal Swedish Geotechnical Institute Proceedings No. 14*.

Hardin, B.O. (1985) Crushing of soil particles, *Journal of Geotechnical Engineering*, ASCE, **111**(10), 1177–1192.

Hardin, B.O. (1987) 1-D strain in normally consolidated cohesionless soils, *Journal of Geotechnical Engineering*, ASCE, **113**(12), 1449–1467.

Hardin, B.O., and Drnevich, V.P. (1972) Shear modulus and damping in soils: measurement and parameter effects, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **98**(6), 603–624.

Hardin, B.O., and Richart, F.E.Jr. (1963) Elastic wave velocities in granular soils, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **89**(1), 33–65.

Harris, W.W., and Finno, R.J. (1994) On the laboratory measurement of time-dependent K_0 behaviour, *Proceedings of 13th International Conference on Soil Mechanics and Foundation Engineering*, New Delhi, **1**, 429–432.

Holtz, R.D., and Jamiolkowski, M.B. (1985) Discussion of "Time dependence of lateral earth pressure by E.Kavazanjian and J.K.Mitchell", *Journal of Geotechnical Engineering*, ASCE, **111**(10), 1239–1242.

Holubec, I., and D'Appolonia, E. (1973) Effect of particle shape on the engineering properties of granular soils, *Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils*, ASTM STP 523, 304–318.

Houlsby, G.T. (1982) Theoretical analysis of the fall cone test, *Geotechnique*,

32(2), 111–118.

Hryciw, R.D., and Dowding, C.H. (1988) CPT and DMT in evaluation of blast-densification of sand, *Penetration Testing 1988*, ISOPT-1, De Ruiter (ed.), **1**, 521–526.

Hryciw, R.D., and Thomann, T.G. (1983) Stress-history based model for G^e of cohesionless soils, *Journal of Geotechnical Engineering*, ASCE, **119**(7), 1073–1093.

Ishibashi, I., Chen, Y.C., and Chen, M.T. (1991) Anisotropic behaviour of Ottawa sand in comparison with glass spheres, *Soils and Foundations*, **31**(1), 145–155.

Ishihara, K. (1985) Stability of natural deposits during earthquakes, *Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, **1**, 321–376.

Ishihara, K. (1993) Liquefaction and flow failure during earthquakes, *Geotechnique*, **43**(3), 351–415.

Ishihara, K., and Okada, S. (1978) Yielding of overconsolidated sand and liquefaction model under cyclic stresses, *Soils and Foundations*, **18**(1), 57–72.

Ishihara, K., and Takatsu, H. (1979) Effects of overconsolidation and K_0 conditions on the liquefaction characteristics of sands, *Soils and Foundations*, **19**(4), 59–68.

Iwazaki, T., and Tatsuoka, F. (1977) Effects of grain size and grading on dynamic shear moduli of sands, *Soils and Foundations*, **17**(3), 19–35.

Jamiolkowski, M., Ghionna, V.N., Lancellotta, R., and Pasqualini, E. (1988) New correlations of penetration tests for design practice. *Penetration Testing 1988*, ISOPT-1, De Ruiter (ed.), **1**, 263–296.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T., and Lancellotta, R. (1985) New developments in field and laboratory testing of soils, *Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering*, San Fransisco, **1**, 57–153.

Jamiolkowski, M., and Manassero, M. (1995) Role of in-situ testing in geotechnical engineering: thoughts about future, *Proceedings of ICE Symposium on Advanced Site Investigation Practice*, London.

Janbu, N. (1985) Soil models in offshore engineering, *Geotechnique*, **35**(3), 241–281.

Jones, R.H. (1961) A preliminary report on the vane test in sands, *Proceedings of The Midland Soil Mechanics and Foundation Engineering Society*, **4**,

163–175.

Joustra, K., and De Gijt, J.G. (1982) Results and interpretation of cone penetration tests in soils of different mineralogical composition, *Proceedings of 2nd European Symposium on Penetration Testing*, Amsterdam, 615–626.

Kallstenius, T., and Bergau, W. (1961) Research on the texture of granular masses, *Proceedings of 5th International Conference on Soil Mechanics and Foundation Engineering*, **1**, 165–170.

Kapoor, J.K. (1985) *Compressibility and Frictional Resistance of a Variety of Sands*, unpublished M.Tech Thesis, Department of Civil Engineering, Indian Institute of Technology Kanpur, India.

Kavazanjian, E., and Mitchell, J.K. (1984) Time dependence of lateral earth pressure, *Journal of Geotechnical Engineering*, ASCE, **110**(4), 530–533.

Koerner, R.M. (1970) Effect of particle characteristics on soil strength, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **96**(4), 1221–1234.

Kokusho, T., Yoshida, Y., and Esashi, Y. (1982) Dynamic properties of soft clay for wide strain range, *Soils and Foundations*, **22**(4), 1–18.

Krizek, R.J., McLean, F.G., and Giger, M.W. (1974) Effect of particle characteristics on wave velocity, *Journal of the Geotechnical Engineering Division*. ASCE, **100**(1), 89–94.

Krumbein, W.C., and Pettijohn, F.J. (1938) *Manual of Sedimentary Petrography*, Appleton-Century-Crofts.

Kuo, C.-Y., and Frost, J.D. (1995) *Quantifying the Fabric of Granular Materials—an Image Analysis Approach*, Report No. GIT-CEE/GEO-95-1, School of Civil and Environmental Engineering, The Georgia Institute of Technology.

Ladd, C.C., Foot, R., Ishihara, K., Schlosser, F., and Poulos, H.G. (1977) Stress-deformation and strength characteristics. *Proceedings of 9th International Conference on Soil Mechanics and Foundation Engineering*, Tokyo, **2**. 421–494.

Ladd, R.S. (1974) Specimen preparation and liquefaction of sands, *Journal of the Geotechnical Engineering Division*, ASCE, **100**(10), 1180–1184.

Ladd, R.S. (1977) Specimen preparation and cyclic stability of sands, *Journal of the Geotechnical Engineering Division*, ASCE, **103**(6), 535–547.

Lade, P.V., Yamamuro, J.A., and Bopp, P.A. (1996) Significance of particle crushing in granular materials, *Journal of Geotechnical Engineering*, ASCE. **122**(4), 309–316.

Lambe, T.W., and Whitman, R.V. (1979) *Soil Mechanics*, John Wiley &

Sons, New York.

Lambrechts, J.R., and Leonards, G.A. (1978) Effects of stress history on deformation of sand, *Journal of the Geotechnical Engineering Division*, ASCE, **104**(11), 1371–1387.

Lee, K.L., and Farhoomand, I. (1967) Compressibility and crushing of granular soils in anisotropic triaxial compression, *Canadian Geotechnical Journal*, **4**(1), 68–99.

Lee, K.L., and Focht, J.A. (1975) Liquefaction potential of Ekofisk tank in North Sea, *Journal of the Geotechnical Engineering Division*, ASCE, **101**(1), 57–72.

Lee, K.L., and Seed, H.B. (1967) Drained strength characteristics of sands, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **93**(6), 117–141.

Leonards, G.A. (1985) Discussion of "Time dependence of lateral earth pressure by E.Kavazanjian and J.K.Mitchell", *Journal of Geotechnical Engineering*, ASCE, **111**(10), 1244–1246.

Marcuson, W.F., and Bieganousky, W.A. (1977a) Laboratory standard penetration tests on fine sands, *Journal of the Geotechnical Engineering Division*, ASCE, **103**(6), 565–587.

Marcuson, W.F., and Bieganouski, W.A. (1977b) SPT and relative density in sands, *Journal of the Geotechnical Engineering Division*, ASCE, **103**(11), 1295–1309.

Mesri, G. (1987) The fourth law of soil mechanics: the law of compressibility, *Proceedings of International Symposium on Geotechnical Engineering of Soft Soils*, Mexico, **2**, 179–187.

Mesri, G. (1993) Ageing of soils, invited lecture, *Simposio Sobre Envejecimiento de Suelos*, Mexico City.

Mesri, G., and Castro, A. (1987) The C_α/C_c concept and K_0 during secondary compression, *Journal of Geotechnical Engineering*, **112**(3), 230–247.

Mesri, G., and Choi, Y.K. (1983) Discussion of "Dynamic properties of soft clays for wide strain range" by T. Kokusho, Y. Yoshida and Y. Esashi, *Soils and Foundations*, **23**(1), 125–127.

Mesri, G., Feng, T.W., and Benak, J.M. (1990) Post densification penetration resistance of clean sands, *Journal of Geotechnical Engineering*, ASCE, **116**(7), 1095–1115.

Mesri, G., and Godlewski, P.M. (1977) Time- and stress-compressibility interrelationship, *Journal of the Geotechnical Engineering Division*, ASCE,

103(5), 417–430.

Mesri, G., and Hayat, T.M. (1993) The coefficient of earth pressure at rest, *Canadian Geotechnical Journal*, **30**(4), 647–666.

Meyerhof, G.G. (1961) The ultimate bearing capacity of wedge-shaped foundations, *Proceedings of 5th International Conference on Soil Mechanics and Foundation Engineering*, Paris, **2**, 105–109.

Mitchell, J.K. (1993) *Fundamentals of Soil Behaviour*, John Wiley & Sons Inc., New York.

Mitchell, J.K., and Solymar, Z.V. (1984) Time-dependent strength gain in freshly deposited or densified sand, *Journal of Geotechnical Engineering*, ASCE, **110**(11), 1559–1576.

Miura, K., Miura, S., and Toki, S. (1986) Deformation behaviour of anisotropic dense sand under principal stress axes rotation, *Soils and Foundations*, **26**(1), 36–52.

Mulilis, J.P., Mori, K., Seed, H.B., and Chan, C.K. (1977) Resistance to liquefaction due to sustained pressure, *Journal of the Geotechnical Engineering Division*, ASCE, **103**(7), 793–797.

Mulilis, J.P., Seed, H.B., Chan, C.K., Mitchell, J.K., and Arulanandan, K. (1977) Effects of sample preparation on sand liquefaction, *Journal of the Geotechnical Engineering Division*, ASCE, **103**(2), 91–108.

Negussey, D., Wijewickreme, W.K.D., and Vaid, Y.P. (1988) Constant volume friction angle of granular materials, *Canadian Geotechnical Journal*, **25**, 50–55.

Oda, M. (1972a) Initial fabrics and their relations to mechanical properties of granular material, *Soils and Foundations*, **12**(1), 17–36.

Oda, M. (1972b) Deformation mechanism of sand in triaxial compression tests, *Soils and Foundations*, **12**(4), 45–63.

Oda, M. (1977) Co-ordination number and its relation to shear strength of granular material, *Soils and Foundations*, **17**(2), 29–42.

Oda, M. (1981) Anisotropic strength of cohesionless sands, *Journal of the Geotechnical Engineering Division*, ASCE, **107**(9), 1219–1231.

Oda, M., Koishikawa, I., and Higuchi, T. (1978) Experimental study of anisotropic shear strength of sand by plane strain test, *Soils and Foundations*, **18**(1), 25–38.

Oda, M., Konishi, J., and Nemat-Nasser, S. (1980) Some experimentally based fundamental results on the mechanical behaviour of granular materials,

Geotechnique, **30**(4), 479–495.

Palmer, S.N., and Barton, M.E. (1987) Porosity reduction, microfabric and resultant lithification in UK uncemented sands, *Diagenesis of Sedimentary Sequences*, Marshal, J.D. (ed.), Geological Society Special Publication No./ 36, 29–40.

Pandey, D.K. (1993) *Effect of Relative Density and Ageing on the Stress-deformation Behaviour of Sands with Different Angularity*, unpublished M.Tech Thesis, Department of Civil Engineering. Indian Institute of Technology Kanpur, India.

Pestana, J.M., and Whittle, A.J. (1995) Compression model for cohesionless soils, *Geotechnique* **45**(4), 611–631.

Pettijohn (1957) *Sedimentary Rocks*, Harper & brothers, Newyork.

Phool Chand (1987) *Effect of Stress History on Strength and Compressibility of Sands*, unpublished M.Tech Thesis, Department of Civil Engineering, Indian Institute of Technology Kanpur, India.

Porovic, E., and Jardine, R.J. (1994) Some observations on the static and dynamic strength of Ham River sand, *Pre-failure Deformation of Geomaterials*, Shibuya, Mitachi and Miura (eds), Balkema, Rotterdam, 25–30.

Poulos, S.J. (1981) The steady state of deformation, *Journal of the Geotechnical Engineering Division*, ASCE, **107**(5), 553–562.

Poulos, S.J., Castro, G., and France, J.W. (1985) Liquefaction evaluation procedure, *Journal of Geotechnical Engineering*, ASCE, **111**(6), 772–792.

Puppala, A.J., Acar, Y.B., and Tumay, M.T. (1995) Cone penetration in very weakly cemented sand, *Journal of Geotechnical Engineering*, ASCE, **121**(8), 589–600.

Quian, X., Gray, D.H., and Woods, R.D. (1993) Voids and granulometry: Effects on shear modulus of unsaturated sands, *Journal of Geotechnical Engineering*, ASCE, **119**(2), 295–314.

Rad, N.S., and Tumay, M.T. (1986) Effect of cementation on the cone penetration resistance of sand: a model study, *Geotechnical Testing Journal*, ASTM, **9**(3), 117–125.

Rahim, A. (1989) *Effect of Morphology and Mineralogy on Compressibility of Sands*, unpublished Ph.D Thesis, Department of Civil Engineering, Indian Institute of Technology Kanpur, India.

Ramamurthy, T. (1969) Crushing phenomena in granular soil, *Journal of Indian National Society of Soil Mechanics and Foundation Engineering*, **6**,

67–86.

Roberts, J.E., and De Souza, J.M. (1958) The compressibility of sands, *Proceedings of ASTM*, **58**, 1269–1277.

Robertson, P.K., and Campanella, R.G. (1983) Interpretation of cone penetration tests, Part I: Sand, *Canadian Geotechnical Journal*, **20**, 718–733.

Rowe, P.W. (1962) The stress-dilatancy relation for static equilibrium of an assembly of particles in contact, *Proceedings of the Royal Society of London, Series A*, **269**, 500–527.

Rowe, P.W. (1971) Theoretical meaning and observed values of deformation parameters for soil, *Proceedings of Roscoe Memorial Symposium*, Cambridge University, 143–194.

Schmertmann, J.H. (1970) Static cone to compute static settlement over sand, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **96**(3), 1011–1043.

Schmertmann, J.H. (1983) A simple question about consolidation, *Journal of Geotechnical Engineering*, ASCE, **109**(1), 119–122.

Schmertmann, J.H. (1984) Closure of the discussions of "A simple question about consolidation by J.H.Schmertmann", *Journal of Geotechnical Engineering*, ASCE, **110**(5), 673.

Schmertmann, J.H. (1987) Discussion of "Time dependent strength gain in freshly deposited or densified sand by J.K. Mitchell and Z.V. Solymar", *Journal of Geotechnical Engineering*, ASCE, **113**(2), 173–175.

Schmertmann, J.H. (1991) The mechanical ageing of soils, *Journal of Geotechnical Engineering*, ASCE, **117**(9), 1288–1329.

Seed, H.B. (1979) Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes, *Journal of the Geotechnical Engineering Division*, ASCE, **105**(2), 201–255.

Seed, H.B., and Peacock, W.H. (1971) Test procedures for measuring liquefaction characteristics, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **97**(8), 1099–1119.

Seed, H.B., Wong, R.T., Idriss, I.M., and Tokimatsu, K. (1986) Moduli and damping factors for dynamic analysis of cohesionless soils, *Journal of Geotechnical Engineering*, ASCE, **112**(11), 1016–1032.

Shahu, J.T. (1988) *Model Studies on Sands with Different Angularity and Mineralogy*, unpublished M.Tech Thesis, Department of Civil Engineering,

Indian Institute of Technology Kanpur, India.

Sheng, Y., and Randolph, M.E. (1994) Effect of density state on the strength and deformation of sands, *Pre-failure Deformation of Geomaterials*, shibuya. MItachi & Miura (eds), Balkema, Rotterdam, 401–406.

Skempton, A.W. (1986) Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation. *Geotechnique*, **36**(3), 425–447.

Sladen, J.A. (1989) Problems with interpretation of sand state from cone penetration test, *Geotechnique*, **39**(2), 323–332.

Sladen, J.A., D'Hollander, R.D., and Krahn, J. (1985) The liquefaction of sands, a collapse surface approach, *Canadian Geotechnical Journal*. **22**, 564–578.

Solymar, Z.V. (1984) Compaction of alluvial sands by deep blasting, *Canadian Geotechnical Journal*, **21**(2), 305–321.

Soydemir, C. (1984) Discussion of "A simple question about consolidation by J.H.Schmertmann", *Journal of Geotechnical Engineering*, ASCE, **110**(5), 669–671.

Stephenson, D. (1979) *Rockfill in Hydraulic Engineering*, Elsevier Scientific Publishing Company.

Stroud, M.A. (1989) The standard penetration test—its application and interpretation, *Penetration Testing in UK*. Thomas Telford, London. 29–49.

Symes, M.J.P.R., Gens, A., and Hight. D.W. (1984) Undrained anisotropy and principal stress rotation in saturated sand, *Geotechnique* **34**(1), 11–27.

Tatsuoka, F., Iwazaki, T., Fukushima, S., and Sudo, H. (1979) Stress conditions and stress histories affecting shear modulus and damping of sand under cyclic loading, *Soils and Foundations*, **19**(2), 29–43.

Tatsuoka, F., Kato, H., Kimura, M., and Pradhan, T.B.S. (1988) Liquefaction strength of sands subjected to sustained pressure, *Soils and Foundations*, **28**(1), 119–131.

Tatsuoka, F., Ochi, K., Fujii, S., and Okamoto, M. (1986) Cyclic undrained triaxial and torsional shear strength of sands for different sample preparation methods, *Soils and Foundations*, **26**(3), 23–41.

Tatsuoka, F. and Shibuya, S. (1992) Deformation characteristics of soils and rocks from field and laboratory tests, *Proceedings of 9th Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Bangkok, **2**. 101–170.

Tavenas, F., and Audy, R. (1972) Limitations of the driving formulas for predicting the bearing capacities of piles in sand, *Canadian Geotechnical*

Journal, **9**, 47–62.

Taylor, D.W. (1948) *Fundamentals of Soil Mechanics*, John Wiley & Sons, New York.

Teachavorasinskun, S., Tatsuoka, F., and Lo Presti, D.C.F. (1994) Effects of the cyclic prestraining on dilatancy characteristics and liquefaction strength of sand, *Pre-failure Deformation of Geomaterials*, Shibuya, Mitachi and Miura (eds), Balkema, Rotterdam, 75–80.

Terzaghi, K. and Peck, R.B. (1948) *Soil Mechanics in Engineering Practice*, John Wiley and Sons, New-York.

Terzaghi, K. and Peck, R.B. (1967) *Soil Mechanics in Engineering Practice*, John Wiley and Sons, New-York.

Terzaghi, K. and Peck, R.B., and Mesri, G. (1996) *Soil Mechanics in Engineering Practice*, John Wiley and Sons, New-York.

Thomann, T.G., and Hryciw, R.D. (1992) Stiffness and strength changes in cohesionless soils due to disturbance, *Canadian Geotechnical Journal*, **29**, 853–861.

Troncoso, J.H. (1994) Time effects on liquefaction potential, *Proceedings of 13th International Conference on Soil Mechanics and Foundation Engineering*, New Delhi, **1**, 189–191.

Vaid, Y.P., Chern, T.C., and Tumi, H. (1985) Confining pressure, grain angularity and liquefaction, *Journal of Geotechnical Engineering*, ASCE. **111**(10). 1229–1235.

Vaid, Y.P., and Sasitharan, S. (1992) The strength and dilatancy of sands. *Canadian Geotechnical Journal*, **29**, 522–526.

Vesic, A.B. (1963) Bearing capacity of deep foundations in sand. *Highway Research Record*, **39**, 112–153.

Vesic, A.B., and Clough, G.W. (1968) Behaviour of granular materials under high stresses, *Journal of the Soil Mechanics and Foundations Division*. ASCE, **94**(3), 661–688.

Whitman, R.V., Miller, E.T., and Moore, P.J. (1964) Yielding and locking of confined sand, *Journal of the Soil Mechanics and Foundations Division*. ASCE, **90**(4), 57–64.

Wong, R.K.S., and Arthur, J.R.F. (1985) Induced and inherent anisotropy in sand, *Geotechnique*, **35**(4), 471–481.

Yamamuro, J.A., Bopp, P.A., and Lade, P.V. (1996) One-dimensional compression of sands at high pressures, *Journal of Geotechnical Engineering*.

ASCE, **122**(2), 147–154.

Yang, N.C. (1970) Relaxation of piles in sand and inorganic silt, *Journal of the Soil Mechanics and Foundations Division*, ASCE, **96**(2), 395–409.

York, D.L., Brusey, W.G., Clemente, F.M., and Law, S.K. (1994) Set up and relaxation in glacial sand, *Journal of Geotechnical Engineering*, ASCE, **120**(9), 1498–1513.

Youd, T.L. (1973) Factors controlling maximum and minimum densities of sands, *Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils*, ASTM STP 523, 98–112.

Youd, T.L., and Perkins, D.M. (1978) Mapping liquefaction induced ground failure potential, *Journal of the Geotechnical Engineering Division*, ASCE, **104**(4), 433–446.

Yudhbir and Rahim, A. (1987) Compressibility characteristics of sands. *Proceedings of 9th South East Asian Geotechnical Conference*, Bangkok, **1**, 483–494.

Yudhbir and Rahim, A. (1993) Image analysis applied to geotechnical engineering. *Proceedings of ASCE Engineering Foundation Conference on Digital Image Processing: Techniques and Applications in Civil Engineering*. Hawai.

Zolkov, E., and Wiseman, G. (1965) Engineering properties of dune and beach sands and the influence of stress history, *Proceedings of 6th International Conference on Soil Mechanics and Foundation Engineering*, Montreal, **1**, 134–138.

Bibliography

Banerjee, N.G., Seed, H.B., and Chan, C.K. (1979) *Cyclic Behaviour of Dense Coarse-grained Materials in Relation to the Seismic Stability of Dams*, Report to California Department of Water Resources, No. UCB/EERC-79/113, EERC, University of California, Berkeley, California.

Dudler, I.V., Durante, V.A., and Smirnov, C.D. (1968) Experience gained in using the penetrometer probe for soil investigation in conjunction with energy-related constructions in the Soviet Union, INFORM-ENERGO, Moscow, Russia.

Dudler, I.V., and Iulin, A.N. (1981) Sand structural characteristics revealed during density measurements using the penetrometer probe, *Hydrogeology and Engineering Geology—Field Methods of Investigation*, Novocherkask, 25–35.

Durante, V.A., and Voronkevich, S.D. (1955) Studies of sand densities for construction purposes, *Annals of Moscow University*, **2**, 131–137.

Ibrahim, A.A., and Kagawa, T. (1991) Microscopic measurement of sand fabric from cyclic tests causing liquefaction, *Geotechnical Testing Journal*, ASTM, **14**(4), 371–382.

Janbu, N. (1963) Soil compressibility as determined by oedometer and triaxial tests, *Proceedings of 3rd European Conference on Soil Mechanics and Foundation Engineering*, Weisbaden, **1**, 19–25.

Kenkyo, K., Teachavorasinskun, T., Tatsuoka, F., Yasuhara, K., and Sato, K. (1991) Effect of cyclic pre-straining on liquefaction characteristics and elastic stiffness of saturated sand, *Proceedings of 46th Annual Conference of JSCE*, **3**, 256–257.

Kolbuszewski, J.J. (1948) An experimental study of the maximum and minimum porosities of sands, *Proceedings of 2nd International Conference on*

Soil Mechanics and Foundation Engineering, Rotterdam, **1**, 158–165.

Kolbuszewski, J.J., and Frederick, M.R. (1963) A contribution towards a universal specification of the limiting porosities of granular materials, *Proceedings of 3rd European Conference on Soil Mechanics and Foundation Engineering*, Weisbaden, **1**, 265–271.

Mogami, T. (1965) A statistical approach to the mechanics of granular materials, *Soils and Foundations*, **5**(2), 26–36.

Omote, S., and Miyamuro, S. (1951) Relationships between the earthquake damage and the structure of ground in Nagoya city, *Bulletin of the Earthquake Research Institute*, Tokyo University, Tokyo, Japan.

Park, C.S. (1992) *Deformation and Strength Characteristics of a Variety of Sands by Plane Compression Tests*, Doctor Eng. Thesis, University of Tokyo.

Powers, M.C. (1953) A new roundness scale for sedimentary particles, *Journal of Sedimentary Petrology*, **23**(2), 117–119.

Simons, D.B., and Albertson, M.L. (1960) Uniform water conveyance channels in alluvial material, *Proceedings of ASCE*, **86**, HY5.

Teachavorasinskun, S. (1989) *Stress-strain and Strength Characteristics of Granular Materials in Simple Shear*, Master Eng. Thesis, University of Tokyo.

Wadell, H.A. (1932) Volume, shape and roundness of rock particles, *American Journal of Geology*, **40**, 443–451.

Zingg, T.H. (1935) Beitrag zur schotteranalyse, Die Schotteranalyse und Ihre. Schweiz Mineral Petrog. Mitt. , **15**, 39–140.